Author:
Dawson, Richard Jonathan

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Performance-based management of flood defence systems

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PERFORMANCE-BASED MANAGEMENT OF FLOOD DEFENCE SYSTEMS

BY

RICHARD JONATHAN DAWSON

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Abstract

Flood defences are economically important safety critical infrastructure systems that need ongoing monitoring and maintenance to ensure their integrity. The aim of the research described in this thesis was to develop new performance-based methodologies that will contribute to improved management of flood defence systems in England and Wales. Performance-based asset management is the process of managing assets in order to optimise their behaviour when evaluated against specified objectives, economic or otherwise. It enables prioritisation and optimisation of investment decisions. It is risk-based in that the costs of interventions can be weighed against their benefits. An initial study identified three key areas of performance-based management in need of improvement: quantitative risk assessment (QRA), probabilistic condition characterisation and decision-support.

QRA provides very important information for performance-based asset management because it can capture the effect which the components of a complex system, individually or in combination, are expected to make to flood risk. However, until recently implementation of QRA in the UK has been inconsistent and limited. Building on previous research a tiered approach to QRA is proposed that provides three stages of increasingly more detailed and accurate assessment to support a range of investment decisions. A high-level method, that has since been applied to estimate a national value of flood risk, is described fully and supported with a case study.

A condition characterisation provides information on the structural performance of a flood defence. Probabilistic methods, whilst being directly compatible with risk-based decision-making have proved to be problematic to apply in practice because of the need for detailed measurements, which may be uneconomical, or indeed impossible to obtain. Available information may not be in a format that is directly applicable to a traditional probabilistic analysis. A new method that can use vague or partial information in a probabilistic analysis to generate imprecise assessments of the conditional probability of failure of a flood defence is described. This is extended to demonstrate how changes in proneness to failure can be monitored using information from defence inspections and model analysis.

A new approach to representing the performance of a flood defence systems is described. The system is represented hierarchically providing an overview of system performance as well as more detailed insights into the performance of specific assets. Performance of sub-systems, including those from a QRA and condition characterisation, is captured by a set of Performance Indicators held in a database. These indicators are projected through value functions reflecting organisational objectives and regulatory standards and are merged to generate a Figure of Merit for the system and each sub-system. Uncertainty in the available evidence is represented and propagated through the hierarchy, providing a commentary on sources and implications of uncertainty. A case study of a flood defence system surrounding a town demonstrates how the methodology provides insights into system performance and can be used to explore different asset management decisions.

to my friends and family

Author's declaration

I declare that the work in this dissertation was carried out in accordance with the regulations of the University of Bristol. The work is original, except where indicated by special reference in the text, and no part of the dissertation has been submitted for any other academic award. Any views expressed in the dissertation are those of the author.

SIGNED: .... DATE:..........................

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Chaos umpire sits,
And by decision more,
embroils the fray,
by which he reigns: next,
him high arbiter,
Chance governs all.

(John Milton, Paradise Lost)

Chapter 1

Introduction

Approximately 10% of the UK population and assets with a capital value of over £200 billion live in the 12,200km^2 that is at risk from flooding by rivers and the sea (Purnell, 2002). These people and their property are protected by over 34,000km of flood defences. Flood defences are therefore economically important safety critical infrastructure systems and need ongoing monitoring and maintenance to ensure their integrity. This is no simple task as:

- the scale of the flood defence infrastructure system means there are a large number of system components in need of management,
- interactions between system components is frequently poorly understood,
- failure mechanisms of flood defences are complex and site specific due to the natural variability in loading and geotechnical conditions,
- monitoring information is scarce and can be expensive to obtain,
- information on system behaviour does not lend itself to being compressed into a single format as it is measured at different resolutions and appears in many formats from vague, imprecise qualitative observations to quantitative and statistical data,
- uncertainties, which may be significant, are expressed in a format appropriate to the type of evidence and these are not always directly comparable, and,
- there may be a large amount of information relating to an investment decision, however it is often only partially relevant, incomplete or conflicting.

Consequently, monitoring and remediation resources can be mis-directed.

An increasing emphasis on strategic planning (DEFRA, 2001b, 2001c, Evans *et al.*, 2002) means decision-makers need to be able to manage and consider large amounts of information describing the behaviour of their system and are therefore facing intense information processing demands (Hall and Davis, 2001). Serious flooding in 1998 and 2000 demonstrated the need for improved management of flood defences (Bye and Horner, 1998, Environment Agency, 2001 and ICE, 2001). Recently the UK government has allocated more resources for improving flood and coastal defence standards (HM Treasury, 2002), however, as in all publicly funded sectors, flood defence managers are coming under increasing pressure to use resources more efficiently and make decisions in a more transparent manner. These factors provide a cogent motive for the novel research aimed at improving decision-support techniques that are described in this thesis.

1.1. RESEARCH OBJECTIVES

A review of flood defence management practices has identified the need for improvement in the three key areas of flood risk assessment, condition characterisation and decision-support techniques. The objectives of this thesis are therefore to:

- justify the need for improvement in these three areas,
- identify suitable techniques that could be developed to improve current methods,
- provide a new flood risk assessment methodology that can be used to assess risk at different levels of resolution so that it can support a multitude of decisions,
- provide a new flood defence condition characterisation methodology that provides an imprecise probabilistic measure of defence performance that takes account of the uncertainty associated with flood defence failure, and,
- provide a new methodology to support decision-making by measuring the performance of defences and the overall performance of the flood defence system.

1.2. FLOOD RISK

Risk is defined as a combination of likelihood and consequences (Adams, 1995). Flood risk is therefore a combination of the likelihood of a flood event and the impacts that would result from this event should it occur (Eiker, 1997). Flooding is considered by some authorities to be the hazard that affects more people than any other (Ward, 1978). Whilst there can be positive environmental impacts resulting from flooding (Thomas, 2002) flood defence engineers are usually concerned with protecting people and property from harm.

Flood risk is recognised as one of the most important indicators of the performance of a flood defence system (DEFRA, 1999 and ICE, 2002). It can be used to identify what may happen in the future, the possible impacts and consequences of these events and their likelihoods. A flood risk assessment provides a rational basis for the development of flood defence management policy, allocation of resources and monitoring the performance of flood mitigation activities. This is because the risk assessment considers the entire flood defence system, its interconnectivity and the costs of intervention or failure whilst incorporating uncertainties associated with the assessment of system behaviour (DEFRA, 2000b, Hall, 2000). Risk assessment is an integral part of risk management which is the process of gathering evidence of system performance, risk assessment, options appraisal and decision-making. Until recently an explicit consideration of risk has been limited to appraisal of major decisions in flood defence infrastructure investment (Hall *et al.*, 1997, Meadowcroft *et al.*, 1997) although there is now a rapid move towards an integrated approach to flood risk management (Hall *et al.*, 2002c).

Flood defence infrastructure management decisions are made at many levels, from a flood defence engineer who is designing a new scheme to the national policy makers in the government who have to allocate annual budgets to the organisations responsible for managing the defences. These decisions can all be informed by a flood risk assessment and it is useful to measure this risk at a multitude of resolutions appropriate to the decision being informed, the scale of the project being undertaken and the available resources.

A tiered flood risk assessment methodology building on the work of Meadowcroft *et al.* (1996) is therefore proposed. A high level risk assessment methodology which is based on only the minimum information available nationally is used to generate national assessments of flood risk, further developing the previously implemented method (Halcrow *et al.*, 2001). More detailed methods require accurate topographical information, crest level surveys in order to take advantage of more sophisticated hydraulic models. More detailed defence information, such as geotechnical properties, enables defence failure probability to be assessed more accurately using more sophisticated reliability methods.

Research described in this thesis focuses on the risk of flooding from flood defence failure. This comprises two components; an estimation of the likelihood of failure of the defence and an evaluation of the consequences associated with this failure. There has been considerable research into improving quantitative flood impact assessments (Parker *et al.*, 1987, Middlesex University, 1990, Penning-Rowsell *et al.*, 1992, Penning-Rowsell *et al.*, 2003) and estimating and modelling flood loads quantitatively (Pugh, 1987, CEHW, 1999, Beven, 2000, HR Wallingford and Lancaster University, 2002). A quantitative flood risk assessment requires estimating failure probabilities of existing flood defences which is the topic of condition characterisation (Section 1.3).

1.3. CONDITION CHARACTERISATION

A condition characterisation provides information on the structural performance of a flood defence. This can be used to identify and prioritise operations and maintenance interventions (Environment Agency, 1996). The present method used in England and Wales ranks the condition of a defence on a scale of 1 ("very good" condition) to 5 ("very poor" condition). This is useful to an extent, but a condition characterisation should provide a description of the structure's proneness to failure, enabling flood defence asset managers to make informed decisions about maintenance, monitoring and replacement strategies. To ensure an efficient allocation of resources, the condition characterisation must be consistent, transparent and auditable. A condition characterisation that supports a risk-based framework is probabilistic, allowing decision-makers to base investment decisions on the proneness to failure of a coastal defence and the consequences, economic or otherwise, of this failure.

Flood defence failure mechanisms are complex and site specific, demonstrating both spatial and temporal variability, for example due to the natural variability in loading regime and geotechnical behaviour. Monitoring information is scarce, can be very expensive to obtain and comes in many different formats. This ranges from a fully probabilistic description of loading to a linguistic description of structural condition.

A move towards a quantitative risk based management framework requires the application of probabilistic techniques to assess the condition of structures. These have only been applied in a limited manner in England and Wales. Appraisal of large projects currently relies on expert judgement to assign defence failure probabilities and deterioration rates (DEFRA, 2000a). Engineers have developed increasingly elaborate methods for the probabilistic assessment of flood defences (CUR and TAW, 1990, Cooke *et al.*, 1993, Oumeraci *et al.*, 2001, Voortman, 2002). Application of these methods to complex infrastructure has often been questioned because they require data that are often not to hand and both difficult and costly to obtain (Parr and Cullen, 1988, Meadowcroft *et al.*, 1996). Probabilistic approaches do not readily include expert judgement which has traditionally formed an important part of condition characterisation. Therefore a probabilistic assessment of defence condition needs to be able to combine vague, imprecise qualitative data, such as evidence from visual inspections, with quantitative and statistical data. The uncertainty associated with this vagueness and imprecision needs to be appropriately represented in the final probabilistic assessment.

This thesis reviews present approaches to condition characterisation and has identified generic needs for a condition characterisation methodology. Methods of handling uncertainty and techniques available to estimate failure probabilities of flood defences are evaluated. The proposed new methodology adapts existing reliability techniques that readily handle uncertainty expressed probabilistically to consider the uncertainty associated with expert judgements. This is illustrated with several examples that show how uncertain evidence can be used and how the deterioration of flood defences can be modelled probabilistically. Probabilistic assessments of structural condition can be used in the flood risk assessment methodology. As more information is made available, a more certain estimate of failure probability is made therefore increasing the accuracy of the flood risk assessment. The ability to model deterioration means that long term changes to flood risk can be predicted.

1.4. DECISION-SUPPORT

Decisions relating to flood defence infrastructure are clearly amenable to a risk based decision-making framework. The flood defence system consists of many components that include flood defence infrastructure, flood warning, drainage, the natural and built environment, physical attributes involved with the water cycle and numerous stakeholders (Kundzewicz and Takeuchi, 1999). The success of components other than flood defence infrastructure, such as flood warning and responding in an emergency is important in providing a flood defence system that performs well. However, the connectivity between the components within the system and the large and varying sources of evidence of their performance makes it difficult for a decision-maker to assemble and use them to support decisions in an auditable manner. A decision-maker may also be constricted by codes of practice, organisational or societal values and other standards. A risk assessment provides useful information on the performance of a flood defence system, and lends itself directly to supporting resource allocation decisions. However, any quantitative risk assessment is inevitably incomplete and will have to exclude information that appears in an inappropriate format or at an unsuitable scale even though it may be relevant to the performance of the defence system. As in other engineering disciplines (SEAOC, 1995 and Coehlo, 1997) a more inclusive measure of performance, which can be thought of as all those aspects of system behaviour relevant to meeting objectives, is a more useful basis for making decisions.

A new approach to representing the performance of flood defence systems has therefore been developed. The systems are represented hierarchically incorporating both high level business processes and operational processes. Performance of sub-systems such as flood defence embankments or flood warning systems is captured by a set of performance indicators, such as flood risk, which are held in a database. Evidence of performance is assembled from all available sources, ranging from monitoring measurements and model studies to expert judgements, analogous cases and accounts of past failures. These performance indicators are projected through value functions reflecting organisational objectives and are merged to generate a figure of merit for each sub-system. Uncertainty in the available evidence is represented and propagated through the evidence hierarchy using Interval Probability Theory, providing a commentary on sources and implications of uncertainty in the decision. A case study demonstrates how the approach can incorporate evidence from the new flood risk assessment method, providing a useful overview of system performance which can be used to explore different asset management decisions.

1.5. ASSOCIATED RESEARCH PROJECTS

The research described in this thesis has been conducted as part of two research and development projects. The first research project entitled 'Condition Monitoring and Asset Management for Complex Infrastructure Systems' (CMAM) has been funded by the Engineering and Physical Research Council (EPSRC) reference GR/M53073/01, the Environment Agency (EA) reference W5A(99)03 and Scottish and Southern Energy (SSE) with contributions in kind from six other industrial partners from engineering consultancies. Whilst many of the principles of the CMAM research could be applied to other fields of engineering, research within this project focused on the flood defence and dams industry. Research contributions described in this thesis which were undertaken as part of the CMAM project are:

(1) an assessment of current asset management practices in the flood defence industry,

(2) an improved method for assessing structural condition of flood defences, and,

(3) a methodology and complimentary software tool to support decision-making.

Research into performance-based management that has resulted from the CMAM project will contribute to development of the EA's new *Performance-based Asset Management System* (PAMS) and to DEFRA's forthcoming Flood and Coastal Defence Project Appraisal Guidance series (FCDPAG6) on performance evaluation.

Improved methods of condition assessment and taking a systems approach to flood defence management naturally compliment the 'Risk Assessment of Flood and Coastal Defence Systems for Strategic Planning' (RASP) project funded by the Department of the Environment, Food and Rural Affairs (DEFRA) and Environment Agency joint research and development programme project W5B(01)02. The research described in this thesis has contributed to the tiered framework and high level methodology for national-scale flood risk assessment, described in Chapter 4, which form part of the RASP project. This level of the methodology is demonstrated with a case study and has been applied nationally under the National Flood Risk Assessment 2002 project, the results of which are given in Appendix G. The methodologies of the intermediate and detailed levels of risk assessment are also described in Chapter 4.

This thesis describes the research undertaken by the author in the context of these two projects. However, it is recognised that these projects are not independent, but are complimentary in their research objectives. Work in this thesis shows how the probabilistic condition characterisation, risk assessment and decision-making methodologies can be integrated thereby enabling flood risk to be considered within the context of system performance. This provides a significant advancement in the field of flood defence management.

1.6. THESIS OUTLINE

Following this introductory chapter, the thesis starts by providing a background and analysis of the problem domain. Flood defence management in England and Wales is reviewed through the study of relevant documents, interviews and site visits to Environment Agency offices. Chapter 3 provides a theoretical background to making a probabilistic risk assessment. The chapter focuses on estimating the failure probabilities of flood defences, but also reviews techniques used to obtain river and coastal loads and estimate damages resulting from inundation. Chapter 4 introduces a new tiered risk assessment methodology that enables risk assessments to be made at a number of different resolutions. The level of the methodology appropriate to making a national scale assessment of flood risk is described in detail. Chapter 5 presents a novel approach to assessing the condition of flood and coastal defences. Reliability methods are used to generate failure probabilities and adapted to enable uncertainties that are described using imprecise information to be incorporated into the assessment. The means to integrate the condition characterisation approach and the risk assessment methodology is also described. Chapter 6 describes new techniques to support performance-based decision-making. The chapter also provides a case study using these techniques that shows how models of a system can be constructed and used to support asset management decisions. The case study also demonstrates how the flood risk assessment described in Chapter 4 and the condition characterisation methodology described in Chapter 5 can be integrated. This provides an overview of system performance that can incorporate measures of flood risk in the context of structural and non-structural performance.

Chapter 2

Background and analysis of problem domain

2.1. OVERVIEW

This chapter provides an overview of flood defence management and aims to set in context the needs and requirements for decision-support tools in England and Wales. The starting point for the development of decision-support tools was a descriptive study of current asset management practices for UK flood defence infrastructure together with a review of practices worldwide. A workshop with representatives from collaborating organisations from within public and private sectors provided an initial impression of the issues and challenges. Subsequent analysis involved a literature review, interviews with experts, site visits and case studies. The outcomes of this process are summarised in this chapter.

This chapter first provides a brief history of flooding in the UK and other countries in Section 2.2. A brief review of flood defence management in several countries is provided in Section 2.3. The structure of flood defence management in England and Wales and key organisations are described in Section 2.4. Guidance documents important to the decision-making process in England and Wales are summarised in Section 2.5 (and described in more detail in Appendix C). A key aspect of prioritising investment in flood defence is the condition characterisation of flood defence assets. Approaches to the condition characterisation of flood defences as applied in England and Wales, Netherlands and the USA are described in Section 2.6. Risk assessment is an important tool for flood risk managers, its benefits and current approaches are described Section 2.7. Section 2.8 identifies the differences and inconsistencies between guidance documents and actual practice. Finally, Section 2.9 identifies areas of flood defence management that would benefit from the novel research in this thesis.

2.2. HISTORICAL CONTEXT

Ward (1978) defined a flood defined as being:

> *"A body of water which rises to overflow land which is not normally submerged"*

This definition encompasses coastal and fluvial flooding, both of which have been experienced in England and Wales.

2.2.1. Flooding in England and Wales

Historically there have been many benefits derived from building on floodplains. Towns often sprung up near river crossing points or natural harbours. Areas of the UK, including the Fens and significant areas of many estuaries, have been reclaimed from the sea and would be underwater for much of the year were it not for drainage and sea defences. Drainage has also helped ensure the productivity of low-lying agricultural land that might otherwise spend a significant period of time waterlogged. Coastal towns derive benefits from trade, industry and tourism. Towns on rivers also benefit from trade and industry, something that was consolidated during the industrial revolution (ICE, 2001). Floodplains have also provided the most productive agricultural land (Purnell, 2002). More recently, housing, industry and infrastructure have encroached onto the floodplain reflecting the fact that it is a valuable resource (Penning-Rowsell and Tunstall, 1996). Agriculture and forestry have intensified, further increasing the demand on rivers to provide water resources and drainage (ICE, 2001).

Historically, floods in England and Wales have been dominated by coastal flooding (Parker, 1985) and early studies by a Royal Commission (1911) focused on this threat. Prior to 1953, coastal defences were constructed to meet local needs, for example protection of seaside resorts (CIRIA, 1986). The North Sea has been described as a splendid sea for storm surges (Heaps, 1983) resulting from both pressure gradients travelling from deep Atlantic waters and wind. One of the most devastating flood events of the last century was the 1953 flood of East Anglia (which also inundated much of the Netherlands) . This flood was caused by a North Sea storm surge dominated by strong wind and took nearly 300 lives and devastated much of East Anglia. After the 1953 flood, the Waverley Committee (1954) recommended that flood defence protection reflect the land use of the protected area. A steady programme of investment meant that the storm surges of 1978 and 1983 resulted in negligible damage and no loss of life in the same area (CIRIA, 1986). However, it is not only the North Sea that poses the threat of coastal flooding in the UK, extreme conditions in the Irish Sea caused flooding in the North West of England in 1977 and 1983 and Towyn in North Wales in 1990 (Welsh Consumer Council, 1992). Teignmouth and Torcross along the South Coast of England have also experienced the wrath of stormy seas in 1975 and 1979 respectively.

Despite these and many other flood events, there has been a common belief that flooding is a rare and infrequent experience in the UK (Fordham and Ketteridge, 1995). However, recent flooding in 1998 and 2000 saw the highest river levels seen in the UK since records began over 270 years ago and provided a reminder of the devastating effects of fluvial flooding. These floods resulted in thousands of properties being flooded and damages running into billions of pounds and prompted several critical reviews (Bye and Horner, 1998, Environment Agency, 2001, NAO, 2001, ICE,2001) into the effectiveness of flood defences and their management in England and Wales and the amount of funding it receives (HM Treasury, 2002).

2.2.2. Flooding as an international problem

Flooding is also a serious problem in other countries the world over. The International Federation of Red Cross and Red Crescent Societies (2001) estimated flooding caused nearly 100,000 deaths between 1991 and 2000 and has been responsible for 58% of deaths related to natural disasters between 1988 and 1997 (Berz, 2000). Flooding is also important economically. Munich Reassurance (1997) estimated that economic damages resulting from flooding were in excess of $130 billion from 1986-1995. The proximity of the countries in Europe means that one weather system can cause inundation in several countries. This was demonstrated in 1953 when the Netherlands was flooded by the same storm as East Anglia in the UK (Pugh, 1987). Flooding of many large cities in Central and Eastern Europe including Dresden and Prague during August 2002 was caused by a slow moving weather front resulting in a week of torrential rain resulting in serious flooding over much of Europe.

On a more global scale movement of warm water in the Pacific Ocean known as El Niño has been shown to cause drought and flooding in different parts of the world (Smith and Ward, 1998). This is a clear demonstration that flooding does not respect administrative boundaries and a reminder of the importance of initiatives such as the European Water Framework Directive (EU, 2000). The United States (Lovelace and Strauser, 1998) and Japan (Kitagawa, 1998) have both experienced serious floods. From 1975 and 1994, economic damages to property and crops in the United States were estimated at being between $27.7 billion and $277 billion whilst between 1600 and 2300 people were estimated to have died because of flooding (Mileti, 1996). In comparison with lesser developed countries, such as Bangladesh, deaths can run into the thousands for a single event. This is often a result of less well developed warning and information dissemination systems. Poor emergency response systems and a lack of medical supplies and sanitation can lead to many more indirect deaths (Smith and Ward, 1998).

2.3. INTERNATIONAL FLOOD MANAGEMENT

International approaches to flood management are usually shaped by the severity and the frequency of flooding, the type of flooding experienced and the economics and culture of the region and its floodplains. This section offers a brief insight into the approach in several countries. The approach to flood defence management in England and Wales is discussed in detail in later sections.

2.3.1. United States of America

Flood management in the USA involves several organisations. The Federal Emergency Management Agency (FEMA) manages and provides funding and in particular aims to minimise casualties for floods that have been declared a major disaster by the US president under The Disaster Relief Act of 1950 (Downtown and Pielke, 2001). The United States Army Corps of Engineers (USACE) plans, designs, builds and operates water resources infrastructure, including flood defences (McKay *et al.*, 1999). Research into improving flood defence management is also performed by the USACE and they have started to employ a variety of quantitative and qualitative risk-based management techniques (National Research Council, 1995, USACE, 1996, 1999b, 2000 and Moser, 1997). 'Alternative' approaches to flood risk management are becoming more popular, Miletti (1999) and Burby *et al.* (2000) have recognised that sustainable natural hazard mitigation is best achieved through effective land-use management. Kunreuther (2000) and Ryland (2000) recognise the importance of insurance as part of a risk management strategy. Hecker *et al.* (2001) identify the benefits to be gained from improvement in emergency management.

2.3.2. The Netherlands

Flood management in the Netherlands is overseen nationally by the Rijkwaterstaat who provide guidance and form national policy. Two thirds of the country (25,000km^2) and the majority of the population are at risk from flooding (Litjens-van loon *et al.*, 2000). The country is divided into eleven provinces that contain a number of dyke rings protecting the low-lying land in polders. 53 Water boards are responsible for the management and maintenance of flood defences in these polders. Prior to 1953, dykes were constructed to the highest known level of flooding. A storm surge in 1953 resulted in the overtopping and failure of a number of sea-dykes, killing 1835 people, destroying 4500 buildings and an economical loss of about 14% of the Dutch GDP (Visser, 1998). Van Dantzig (1956) recommended the consideration of economics in decision-making and since the 1953 floods, the level of protection has been risk-based with the most densely populated and economically important areas receiving protection against the 1 in 10,000 year flood event (Jak and Kok, 1999). After the 1996 Flood Defence Act, these became set down in probabilistic terms (Voortman *et al.*, 2001) and each defence is required to be assessed for safety every 5 years (De Looff, 1998). Increasingly risk-based and probabilistic techniques are being employed to aid design, maintenance and assessment (CUR and TAW, 1990, Cooke *et al.*, 1997, Jonkman *et al.*, 2002, Voortman *et al.*, 2002). Without flood defences, two thirds of the Netherlands would be flooded, this vulnerability has ensured large amounts of investment in the past on construction and maintenance of defences to a high standard rather than on developmental control, warning, emergency repair or insurance (De Ronde, 1998). A result of the high vulnerability of the Netherlands to flooding is that floodplain management is not a feature of flood management (Penning-Rowsell and Tunstall, 1996).

2.3.3. Bangladesh: A less developed country

A brief review of flood management in the USA and the Netherlands suggests that more developed countries are employing ever more sophisticated tools and techniques. In lesser developed countries, lack of funds means these are less easily implemented. Bangladesh, for example, takes an alternative approach to flood management. Because of its low elevation, over two thirds of the country is at risk of flooding. This risk is compounded by increased urbanisation, however, due to limited resources and lack of knowledge many households are unprotected against flooding

(Arambepola, 2002). Between 1950 and 1988, 25 major flood events resulted in over 60% of the country being inundated (Khalil, 1990). There is no one dominant flooding mechanism, with flash floods, rainwater flooding, river flooding and coastal surges all being a major threat (Pugh, 1987 and Smith and Ward, 1998). Bangladesh has a Water Development Board with responsibility for flood defence, however, whilst they have previously implemented structural solutions to flooding, the success of these has been questioned (Thompson and Sultana, 1996). Much of the funding for flood management comes from the international aid community due to the relative regional poverty. Instead, an approach to 'living with floods' has been suggested that does not rely on enormous amounts of infrastructure that a poor nation such as Bangladesh can not hope to maintain (Khalequzzaman, 1994). Such an approach would attempt to minimise the danger and disruption of flooding, by drawing a distinction between frequent but benign floods that often bring positive consequences (such as soil fertility) and the infrequent but disastrous events (Rumi, 2002).

2.4. STRUCTURE OF FLOOD DEFENCE MANAGEMENT IN THE UK

The structure of flood defence management is complex and rapidly evolving. The responsibilities for flood defence in the UK are laid out in several Acts of Parliament described in Section 2.4.1 below. These are enacted by several organisations, the most important of these is the Environment Agency which has a supervisory role over all matters related to flood defence in England and Wales*. This section describes the organisations that are involved in flood defence management in England and Wales and the legislation that governs them.

2.4.1. Legislation

Four key Acts of Parliament cover flood defence which define responsibilities, administrative processes and funding powers for engineering works. A summary is shown in Figure 2.1.

Land Drainage Act

The Land Drainage Act of 1991 is concerned with defence against water and covers internal drainage associated with all ordinary watercourses, as well as considering coastal flooding. The Act requires that private owners of watercourses maintain them to an appropriate standard. Operational responsibility for coastal works lies with local authorities under supervision from the EA. The authorities must apply for approval from DEFRA or the Welsh Office who may also provide grant aid. The Act also defines a number of key terms:

- *Main rivers* are watercourses designated as such on main river maps and are generally the larger arterial watercourses.
- *Ordinary watercourses* are all those watercourses that are not designated as main river.

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* Whilst aspects of flood defence management in Scotland and Northern Ireland are similar to that in England and Wales, it is governed by alternative arrangements not specifically addressed in this thesis.

- *Critical ordinary watercourses* are ordinary watercourses which the Environment Agency and other operating authorities agree are critical because they have the potential to put at risk from flooding large numbers of people and property.
- *Sea defences* are measures to help prevent flooding from the sea.
- *Coastal protection* are measures to protect the land against erosion and encroachment by the sea.
- *Coastal defence* is an overarching term that includes both sea defence and coast protection.

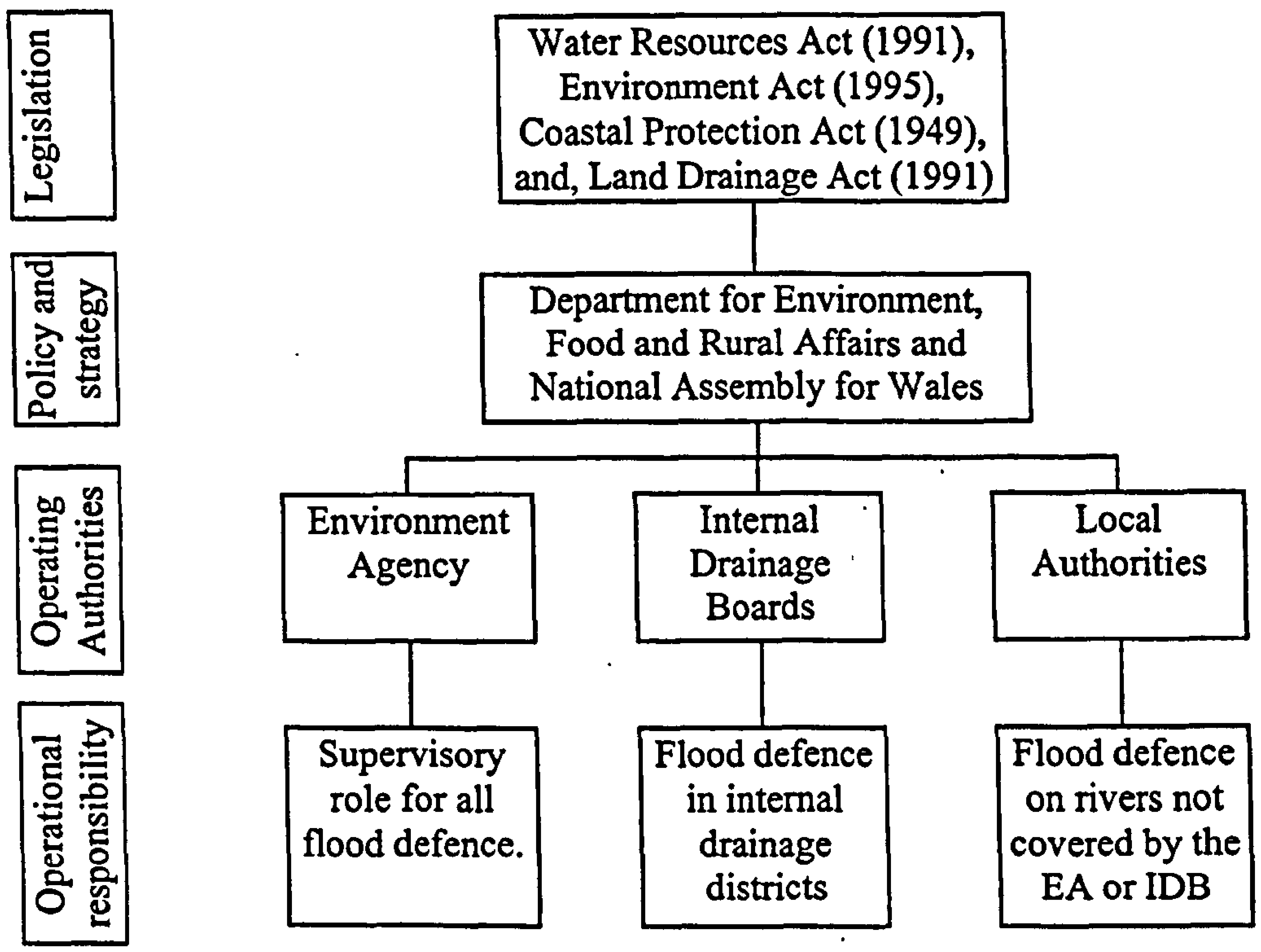

Figure 2.1 Key legislation, roles and responsibilities for flood defence in England and Wales (adapted from NAO, 2001)

Water Resources Act

The Water Resources Act of 1991 deals with the drainage of land and provision of flood warning systems. The drainage of land covers main rivers and provision of sea and tidal defences. Operational responsibility lies with the Environment Agency with grants from DEFRA and the Welsh Office. The Environment Agency is also now required to provide the same service for critical ordinary watercourses.

Environment Act

The Environment Act of 1995 established the Environment Agency and their Scottish counterpart the Scottish Environmental Protection Agency. The Act defines the roles of the parties involved in flood defence, and section 6(4) defines the role of the EA stating that:

> *"....the (Environment) Agency shall in relation to England and Wales exercise a general supervision over all matters relating to flood defence."*

The Act also defines strategies for other environmental issues such as mines, national parks, air quality and waste management.

Coastal Protection Act

The Coastal Protection Act of 1949 governs the funding, construction and operation of measures to protect the land from erosion or encroachment by the sea. Operational responsibility lies primarily with local authorities. Approval is required from DEFRA or the Welsh Office for construction of coast protection works. These organisations also provide grant aid for large projects.

Other Acts and Regulations

There are other Acts and statutory instruments that can have an impact on river and coastal engineering projects. These include Health and Safety at Work Act 1974, Town and County Planning Regulations 1988, Conservation Regulations 1994 and the Flood and Environmental Protection Act 1985 which governs the deposition of anything below Mean High Water Springs tide level.

2.4.2. Organisations involved in flood defence management

The key organisations involved in flood and coastal defence management in England and Wales, their roles and interactions are described below.

DEFRA (formerly MAFF) and The National Assembly for Wales

The Department for Environment, Food and Rural Affairs (DEFRA) oversees the Environment Agency with respect to flood defence in England, whilst in Wales this responsibility lies with the National Assembly for Wales. DEFRA has responsibility for establishing policy, issuing national guidance and paying capital grants to fund large flood and coastal defence projects. In 1999, DEFRA outlined its high level targets for flood and coastal defence (DEFRA, 1999) and in 2001 a joint research and development programme was started in conjunction with the Environment Agency following recommendations by Penning-Rowsell (1998).

The Environment Agency

The Environment Agency came into being on the 1st April 1996. It is non-departmental public body that was established under the provisions of the 1995 Environment Act. It is an amalgamation of the National Rivers Agency, HM Inspectorate of Pollution and the waste regulation arm of the Local Authorities. The Environment Agency have a supervisory role for all flood defence matters in England and Wales, but also have a duty to conserve and enhance the natural environment. The EA have jurisdiction over main rivers, critical ordinary watercourses and sea defence works. The EA is responsible for managing 34,000km of defences, that defend just under 10% of the population and 12,200km^2 of England and Wales from flooding.

The EA has a tiered organisational structure in order to provide national consistency in policy provision, but also maintain local accountability in service provision (Parish, 1998). The head office based in Bristol is responsible for policies, standards and ensuring a consistent approach. There are eight regional offices which are arranged on the basis of river catchment areas for water management (although the boundaries are often different for pollution and waste control purposes). Each region is further split into three or four areas providing a total of 26 area offices over England and Wales.

In response to the Bye Report (Bye and Horner, 1998) the EA restructured its flood management activities in order to deliver its core services which are (Harman *et al.*, 2002):

(1) Strategic Planning to assess medium and long term management options for the flood defence system,
(2) Operating a flood warning system to forecast and disseminate warnings,
(3) Managing emergency response in the case of a flood,
(4) Managing capital programme of improving, replacing or constructing defences,
(5) Regulating development in the floodplain and along watercourses,
(6) Operating and maintaining flood defence infrastructure and channel capacity,
(7) Flood risk mapping to provide an indication of the level of risk within a floodplain, and,
(8) Researching and developing new tools and techniques needed for effective flood management.

Whilst the services are delivered at a local, regional and national level, the processes enacted to deliver these services will vary at each level within the organisation. For example, policy and best practice for flood warning are disseminated to regional offices that oversee the implementation of these policies across the region. However they are predominantly implemented at an area level where the staff are responsible for disseminating the warning to the public.

Regional and Local Flood Defence Committees

Flood defence committees were set up to raise funds from local authorities and to agree flood defence programmes recommended by the Environment Agency. Local committees have delegated powers from the regional committee to raise funds for their area.

Internal Drainage Boards

There are 235 Internal Drainage Boards (although they operate as 65 consortia). These Boards have jurisdiction over ordinary watercourses in an Internal Drainage District. The Boards secure funding from local land owners and Local Authorities. They also pay levies to the EA to fund work on main rivers that protects internal drainage districts.

Inland Local Authorities

These authorities (usually councils) have jurisdiction over ordinary watercourses except in an Internal Drainage District. They pay levies through flood defence committees to fund alot of the Environment Agency's flood defence work. Local authorities (in conjunction with other organisations such as the emergency services) are responsible for the implementation of emergency plans in the event of serious flooding.

Maritime Local Authorities

These authorities have jurisdiction over coast protection (protecting of the coast from erosion, rather than defending low lying land from flooding) and ordinary watercourses except where they are in an Internal Drainage District.

2.5. FLOOD DEFENCE MANAGEMENT PROCEDURES AND GUIDANCE

It is important to understand not only the organisation involved in flood defence management and their responsibilities, but also the existing management framework. This enables the needs of the decision-makers, which are discussed in Section 2.9, to be better identified.

Flood defence management procedures in England and Wales are laid out in several key documents published by DEFRA and the EA, it is also heavily influenced by several well used manuals and other forms of guidance. Some of the procedures in this guidance are very detailed and so a fuller description is provided in Appendix C.

2.5.1. Flood Defence Management Manual

The *Flood Defence Management Manual* (FDMM) (Environment Agency, 1996) provides a framework for prioritising and justifying maintenance decisions and small (<£50,000) capital projects of river defences. These procedures are currently being reviewed. The FDMM acts as a guidance document for the procedures required by the Flood Defence Management System (FDMS). The FDMS is a database that also includes some pre-programmed routines of prioritisation and justification methods outlined in the FDMM. Data from the FDMS has now been migrated into the EA's new *National Flood and Coastal Defence Database* (NFCDD) which does not include any in-built decision-support functionality. The aim of the NFCDD is to provide a definitive source of all data on flood and coastal defences (including those not managed by the Environment Agency) to help make better informed decisions (Linford *et al.*, 2002). Asset management procedures within the FDMM are supported by the Condition Assessment Manual (Glennie *et al.*, 1991) which is used as a guide to help flood defence inspectors assess defence condition. The present methodology for condition assessment is discussed in detail in Section 2.6.1 and a new methodology is proposed in Chapter 5.

The principal items of information required by the FDMM are as follows:

(1) A description of defence condition on a five point scale from 1 (Very Good) to 5 (Very Poor) which summarises the inspection information on the state of the existing asset, guidelines for which are laid out in the Condition Assessment Manual.

(2) Surveys that evaluate the assets in the flood risk area *i.e.* the potential consequences of flooding.

(3) Data about previous floods and predictive models of flooding are gathered (although this can often be limited) so that estimations can be made of the likelihood of flooding.

(4) Cost estimates are required to compare alternative implementation and scheme options.

This information is used to calculate a value of the Standard of Service (SoS) for a given length of river. The SoS is a measure of the expected damage per kilometre of river per year.

The decision outcomes from the FDMM are either:

- 'Do nothing'
- Improve present SoS (capital scheme)
- Perform structural repairs (periodic maintenance)
- Cleaning, mowing *etc* (routine maintenance)

Routine maintenance is prioritised on the basis of the service provided (using the SoS number as a prioritisation indicator) and periodic maintenance is provided using the condition assessment. Capital projects, if appraised using the FDMM, consider economic, environmental and social factors as well as the SoS and integrity of any defences.

2.5.2. Management Plans

Shoreline Management Plans (SMPs) (MAFF, 1995, DEFRA, 2001c) lay out procedures for developing a long term strategy for sustainable coastal defence within coastal sediment cells, taking account of natural coastal processes and human and other environmental influences and needs. SMPs are prepared by coastal groups that include representatives from the EA and Maritime Local Authorities. There are a number of generic policies available to the shoreline manager.

- *Hold the existing defence line* by maintaining or changing the standard of protection. This policy should cover those situations where works or operations are undertaken in front of the existing defences (this includes interventions such as beach recharge, rebuilding the toe of a structure, the construction of offshore breakwaters) to improve or maintain the standard of protection provided by the existing defence line. Policies that involve operations to the rear of existing defences (for example, construction of secondary floodwalls) should be included under this policy when they form an integral part of maintaining the current coastal defence systems.
- *Advance the existing defence line* by constructing new defences seaward of the original defences. Note that use of this policy should be limited to those management units where significant land reclamation is considered.
- *Managed realignment* by identifying a new line of defence and, where appropriate, constructing new defences landward of the original defences.

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- *Advance the existing defence line* by constructing new defences seaward of the original defences. Note that use of this policy should be limited to those management units where significant land reclamation is considered.
- *Managed realignment* by identifying a new line of defence and, where appropriate, constructing new defences landward of the original defences.

- *Limited intervention* by working with natural processes to reduce risks while allowing natural coastal change. This may range from measures which attempt to slow down rather than stop coastal erosion and cliff recession to measures that address public safety issues such as flood warning systems. In other cases, measures might be undertaken to maintain the natural standard of defence on accreting shorelines, for example, managing blow-outs on advancing dune systems.
- *No active intervention,* where there is no investment in coastal defence assets or operations, *i.e.* no shoreline management activity.

A high technical content is required for the SMP. Part of the planning process requires the consultation of all stakeholders including the general public. The forthcoming *Catchment Flood Management Plans* (HR Wallingford *et al.,* 2001) will provide similar guidance for river catchments. CFMPs will be supported by the *Modelling and Decision-Support Framework* (MDSF) that automates part of the process, in particular river modelling and identification of impacts (Wicks *et al.,* 2002, Ramsbottom *et al.*, 2002).

2.5.3. DEFRA Project Appraisal Guidance

The DEFRA *Project Appraisal Guidance* (PAG) series (DEFRA 2000a, 2000b, 2000c, 2001a, 2001b) define the procedures for appraising capital works. The series consists of six volumes which provide guidance on different aspects of project appraisal. PAG1 (DEFRA, 2001a) provides an overview of the project appraisal process and guidance on integrating the key elements. *Strategy Plans* (Coastal Defence Strategy Plans and Sub-catchment Plans) (DEFRA, 2001b) are more rigorous studies on achieving the aims of the Management Plans within smaller areas. PAG3 (DEFRA, 2000a) gives guidance on identifying methods for valuing costs and impacts in monetary terms and sets out a recommended decision process, based on economic values. Of particular note is the use of probabilistic discounting over the life of the plan. Probabilities of defence failure and degradation rates over the plan duration are assigned using expert judgement. PAG4 (DEFRA, 2000b) gives guidance on managing risk and uncertainty. Risk assessment methods are reviewed and guidance on probabilistic assessment is provided. PAG5 (DEFRA, 2001c) provides guidance on appraisal of environmental consequences. Aside from general guidance on best practice and environmental impact assessment, methods for assigning costs to environmental losses are given. PAG6 is unpublished but will provide guidance on performance evaluation and covers the appraisal of policies and plans as well as individual schemes in order to determine their effectiveness and efficiency in delivering the original aims and objectives. The guidance will also provide a framework for integrating lessons learned into future project management.

2.5.4. Additional guidance

Since the early 1990s, there have been a number of initiatives in regard to non-statutory plans that deal in particular with flooding and coastal issues. Many of these contain policies and proposals that have land use planning implications, some directly involve flood defence managers, whilst others may impact only indirectly.

The *Planning Policy Guidance Note 25* (PPG25) (ODPM, 2001) is aimed at providing guidance to local planning authorities in order that they use their existing powers to guide, guide, regulate and control development in accordance with government guidelines. PPG25 aims to raise awareness to local authorities about the issues involved with development in flood risk areas and ensure that as well as local issues, catchment or coastal cell scale issues are also addressed. *Estuary management plans* (EMP) focus on ensuring a sustainable use of estuaries and are prepared by all major stakeholders. *Harbour management plans* (HMP) are similarly produced for harbours. *Coastal habitat management plans* aim to develop sustainable coastal defence strategies in areas of important wildlife. *Local Environment Agency Plans* (LEAP) are produced by the Environment Agency on a catchment basis to develop a more holistic long term approach to achieving all its aims with respect to flood defence and other issues such as water quality, fisheries and recreation. *Water Level Management Plans* (WLMP) identify the water level requirements for a range of activities such as flood defence, agriculture and conservation. *River basin management plans* are required by the EU Water Framework directive (EU, 2000) which sets out the objectives of the water bodies in the river basin and how they will be achieved. Other plans that may have an impact on flood defence management are *Community strategies, Heritage Coast Management Plans, Biodiversity Action Plans* (BAP), *Integrated Coastal Zone Management Plans* (ICZM).

Guidance in England and Wales also comes from other organisations such as the Construction and Industry Research and Information Association (CIRIA) in the form of manuals. Some of the more important manuals provide guidance on river bank protection (Hemphill and Bramley, 1989), rock use in coastal engineering (CIRIA and CUR, 1990), seawall design (Thomas and Hall, 1992), the beach management manual (CIRIA, 1996) and guidance on general risk assessment (CIRIA, 1996). The USACE Coastal Engineering Manual (USACE, 2002) and the Environment Agency's overtopping manual (HR Wallingford, 1999) are also frequently used design tools. In addition to this the British Standards Institution publishes codes, such as BS6349 (British Standards, 1989-2000) that provides guidance on construction of marine structures.

2.6. CONDITION CHARACTERISATION

A condition characterisation is an important indicator of the structural performance of a fluvial or coastal defence. As described in Section 2.5, they are currently used to justify investment decisions in England and Wales. Three differing approaches to condition characterisation are described in the following sections.

2.6.1. England and Wales

The present condition characterisation methodology used in England and Wales by the Environment Agency ranks a flood defence between 1 ("very good") and 5 ("very poor"). This score is based on visual inspection of the defence by comparison to standard photos, using linguistic descriptions of condition set out in the Condition Assessment Manual (Glennie *et al.*, 1991). An example page from the manual is shown in Figure 2.2 for a bank slope of "good" condition. At this basic level no precise measurements are made. Every flood and coastal defence in England and Wales is required to have been assessed using this method.

Bank Slopes

	1 Very Good	2 Good	3 Fair	4 Poor	5 Very Poor
Condition		●			

Specific Description:
In reasonable condition. Minor defects, minor routine maintenance required, stable side slope well vegetated, stone revetment at toe in good condition.

General:
Minor, non-urgent defects. Minor routine maintenance is only work required. In reasonable condition but with some increase in maintenance needed. Slight defect, not more than 5% of length or area affected.

Figure 2.2 An example page from the Condition Assessment Manual showing a bank slope of "good" condition (Glennie et al., 1991)

More detailed assessments as recommended by the National Rivers Authority (1991) and DEFRA (1999) have only partially been implemented in England and Wales. The approach considers failure modes and has a tiered format, with more frequent and detailed levels of inspection being required on identification of a problem.

An initial 'baseline survey' gathers all available data, where possible using as-built design information. Frequently this will not be available and therefore additional investigations will be required. The information gathered for this more detailed investigation should include:

- topographic surveys,
- channel or beach profiles,
- bathymetric surveys,

- aerial photos,
- land ownership surveys,
- visual condition and photographs,
- outfall and culvert surveys,
- structural surveys,
- diving inspections,
- geotechnical investigations, and,
- mechanical and electrical inspections.

This information is then assessed in the context of the level of protection needed from the asset. Particular consideration should be given to the level of the structure, its strength under normal and extreme loading conditions and ability to pass flood flows. The level of detail to which these investigations are undertaken is risk-based and considers the likely consequences of failure.

'General' inspections are carried out by in-house staff every 6 months to two years. 'Principal' inspections are carried out by engineers in place of a general inspection once every three or more years. These inspections are used to monitor any deviation in performance from the 'baseline' inspection, and to identify the presence of any defects and the need for intervention. The frequency of inspection is risk-based, considering the importance of the assets in the system and past experience of its behaviour. The main difference between the two types of inspection is that principal inspections are more thorough, and involve making measurements as opposed to a solely visual inspection. Whilst any defects and their consideration in relation to failure modes are expressed qualitatively, the outcome of the inspections is summarised using the condition grade described previously. General or principal inspections should also be undertaken after an extreme event.

The NRA (1990) report recommends that crest level surveying should be performed at least once every ten years, and more frequently if settlement is quite fast. Thorough structural inspections are required if general inspections conclude there is a cause for concern. These are based on specific consideration of failure modes and analysis of whether the structure is able to meet its performance objectives. Underwater inspections may also be necessary. The aim of these procedures is to monitor significant parameters at regular intervals, allowing asset managers to identify trends in asset behaviour. One off incidents of damage are therefore not incorrectly identified as being part of progressive defence failure.

The condition grade is used in the prioritising of maintenance works, as described in section 2.5.1 (and in more detail in Appendix C), which outlines the methods within the Flood Defence Management Manual.

2.6.2. United States of America

In the US, the Army Corps of Engineers uses a scoring system to rank defences from 0 ("failed") to 100 ("excellent condition"). The condition index considers the defence's structural integrity and its ability to fulfil its functions (Oliver *et al.*, 1997). Actual approaches can vary between defence types, but the overall condition index, $CI_{combined}$, for a defence combines functional and structural ratings for a multitude of damage patterns. A damage pattern is a measurable indicator of the proneness of a structure to a particular failure mode. For example, measuring crest height provides an indication of proneness to overtopping and overflow. A weighting is applied to emphasise the relative importance of damage patterns. A final value for the condition index for n damage patterns can therefore be obtained using Equation 2.1 (McKay *et al.*, 1999).

$$CI_{combined} = \sum_{i=0}^{n} (W_i)(C_i),\ 0 \le CI_{combined} \le 100 \qquad (2.1)$$

where W_i is a weight that represents the relative importance of the damage pattern score, C_i to the overall structural integrity. The weighting factors in Equation 2.1 are rarely linear and so an adjustment factor, AF_i, an example of which is shown in Equation 2.2 (McKay *et al.*, 1999), is applied to account for the increasing dominance of a particular damage pattern, C_i, as it approaches zero. The weights, W_i, are re-normalised accordingly.

$$AF_i = 8 - 7\left(\frac{C_i - 40}{30}\right),\ 40 \le CI_i \le 69 \qquad (2.2)$$

This example shows how as the damage pattern score, C_i reduces from 69 to 40, the value of W_i is multiplied by up to 8 times, representing the increasing likelihood that the structure will fail by the corresponding mode. If a particular damage pattern is deemed to be 'critical' by an expert, the value of $CI_{combined}$ is set to the value of the C_i for the critical damage pattern.

The Condition Index system provides an indicator of a structure's performance, and is used to justify increased funding and an aid for prioritising flood defence works.

2.6.3. The Netherlands

The approach to condition characterisation in the Netherlands is tiered, consisting of three levels of progressively more accurate assessment. The output of an assessment is a linguistic statement classifying a segment of defence as being 'good', 'satisfactory', 'unsatisfactory' or of 'uncertain' stability for the next five years (De Looff and Van der Meer, 1998). All dykes are analysed at a basic level and then if classified as uncertain, more accurate assessment techniques are employed.

Condition assessments of flood defences in the Netherlands focus on the design level. The assessment of an asset is based on it fulfilling the criteria to meet the design loads. The current safety standards require that the failure probability of a dyke is 10% for an event with an exceedance probability of 0.01 in 100 years (Pilarczyk, 1999). The frequency of condition assessment is five years.

A dyke system in the Netherlands can span great lengths. Due to spatial and temporal variations of hydraulic loading, revetment type, structural dimensions and soil properties, a dyke system will show a varying degree of dependency along its length (CUR and TAW, 1990). For ease of analysis, the dyke system is divided into sections based on the previously listed properties. To meet the legal requirements, flood defence structures must be of sufficient height, stability or in the case of dunes systems or beaches have a suitable volume or profile.

Basic Assessment

The first level of assessment considers limited evidence of condition and failure modes and applying this to various rules of thumb, derived from experience and experimental results, an assessment of stability is made (Stoutesdijk *et al.,* 1998). This assessment is based on correlating visual observations of the slope, core, armour layer size, cover layer thickness, filter laws and freeboard with these rules. For example experience has shown that a slope with a shallower gradient than 1:4 will not fail and the dyke can therefore be immediately classified as 'safe' with respect to slope failure.

Detailed Assessment

This level of assessment considers the dyke's stability with greater accuracy than the previous assessment, allowing for more structures to be classified as safe or unsafe by reducing the uncertainty associated with the condition assessment. Measurements are taken of the design parameters of a dyke. These are then assessed using the current design criteria (CUR and TAW, 1991 and TAW, 1999) to assess whether the structure satisfies these requirements. This level involves a more specific consideration of the values of key parameters. More parameters are evaluated than in the basic assessment and software is used to support the assessment.

A structure is classified as 'safe' if it meets all the current design standards (for example hydraulic gradient less than critical hydraulic gradient). To classify a structure as 'unstable' it must clearly fail to meet design criteria (for example hydraulic gradient is greater than 1.5 times the critical hydraulic gradient). Structures still not definitively shown to be either 'satisfactory' or 'unsatisfactory' are classified as 'uncertain' and undergo an advanced assessment.

Advanced Assessment

The advanced level of assessment requires a much more thorough inspection of the structure involving numerical model simulations as well as in-situ tests. As much data is collected as possible at this level of analysis with sampling at intervals of between 100 and 250 metres along the length of the dyke.

To test the permeability of the cover and filter layers, parts of the revetment are removed to study the filter and clay layer underneath, searching for silt between the cover-layer and filter layer. The efficiency of the filter layer is also tested. Consideration of the filter layer's permeability is important (Bezuijen and Kruse, 1998) because this will influence the likely failure mode. Block pull-out tests allow the calculation of inter-block clamping forces for block revetments. The blocks would be selected for the tests if after visual inspection they appeared to be looser than surrounding blocks. Analysis of the clay layer is done by digging a trench to get an exact profile of the dyke. This also allows the integrity of the clay to be considered so that a judgement of the residual strength can be made. However, this analysis is not universally favoured because of the damage caused to the dyke by the invasive investigation.

Final tests involve the use of the ZETSTEEN (CUR and TAW, 1995) numerical model. Monitoring of the phreatic line, wave pressure and filter layer pressure over normal tidal conditions and storm conditions allows the revetment loads to be analysed over a multitude of conditions. A final assessment of 'satisfactory' or 'unsatisfactory' is then made.

2.7. RISK

This section introduces the concept of risk. The process of risk management is introduced and a number of methods available for assessing risks are reviewed. The benefits of risk assessment as a tool to support decision-making are discussed in the context of the requirements of a flood defence manager.

2.7.1. Definition of Risk

The British Standards Institution (1991) provide a technical definition of risk.

> *"Risk is the combination of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequences of the occurrence"*

A hazard is defined as a situation that could occur that has the potential for human injury, damage to property, damage to the environment, or economic loss. Whilst the term 'risk' can refer to risks at all stages of a flood defence project such as design risk, procurement risk and construction risk, this thesis focuses on the risk to the natural and built environment from flooding. Flood risk is therefore a measure of the likelihood of a flood event and its economic, social or environmental impacts, which is formally defined by Equation 2.3:

$$\textit{Flood Risk} = \textit{Probability of flooding} \times \textit{Consequence of flooding} \qquad (2.3)$$

How this definition is interpreted will often vary as different stakeholders will have alternate perspectives of flood risk. National policy advisors will usually be interested in annual statistics for the whole country, whereas individuals in a floodplain will be more concerned about their personal risk (although these are not necessarily mutually exclusive). As seen in Section 2.2 the impacts of flooding vary between countries and it is only natural for people from different areas to weigh the economical, social and environmental impacts differently to each other and sometimes national policy makers.

2.7.2. Risk management

Risk management framework

Risk management aims to facilitate the assessment and mitigation of undesirable outcomes (such as flooding) by providing a common framework for identifying uncertainty and comparing different intervention options. The generic risk management framework (Figure 2.3) includes the process of risk assessment which is described in Section 2.7.3.

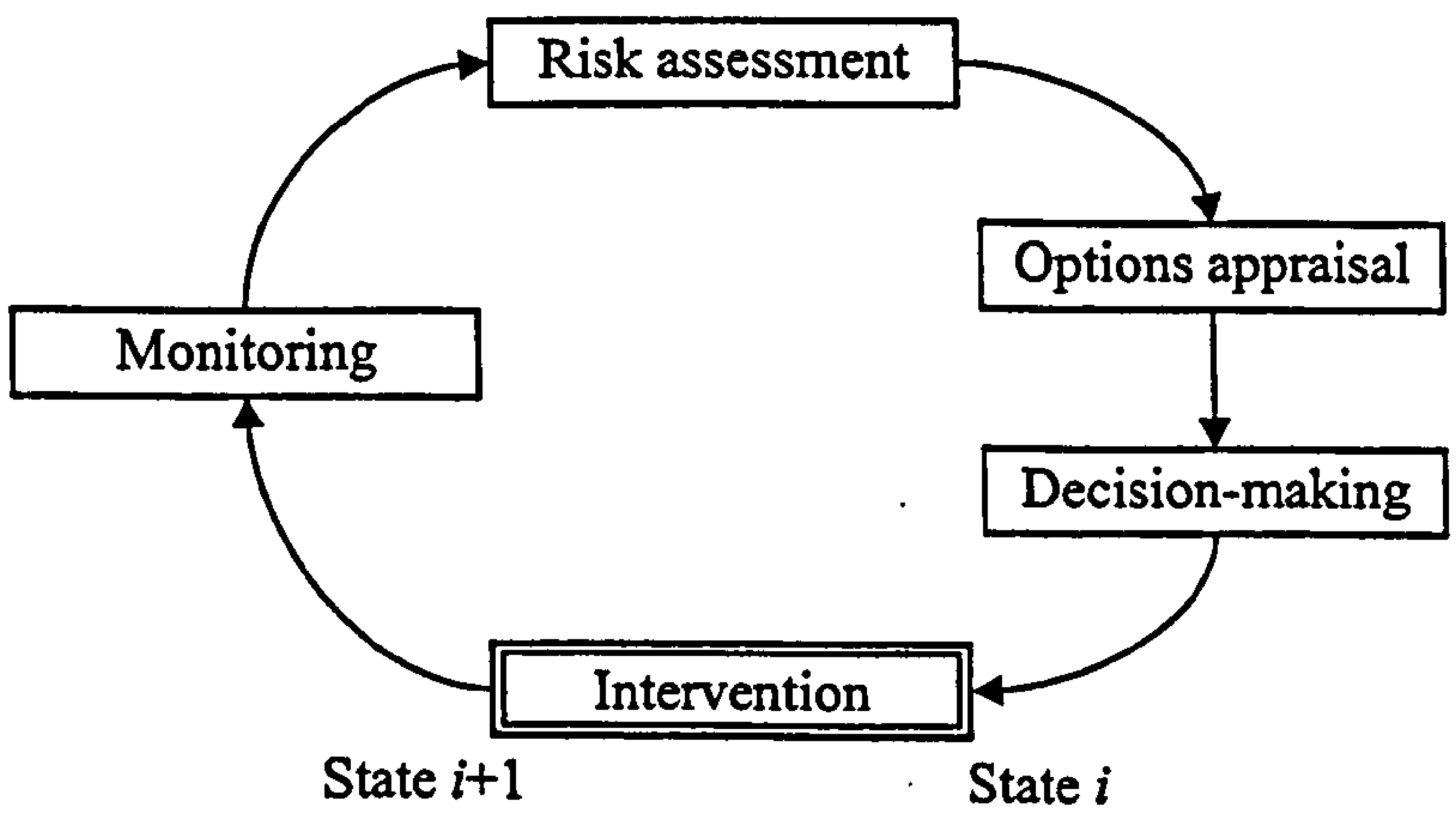

Figure 2.3 The generic risk management cycle (adapted from HM Treasury, 2001)

The flooding system

Before risk management can begin, the scope of the system must first be identified. Flooding may have positive or negative impacts on any part of the natural and built environment. Integrated flood risk management incorporates these different aspects of the flooding system and their influences on each other. Key elements of the flooding system include (Penning-Rowsell and Tunstall, 1995, Smith and Ward, 1998, Kundzewicz and Takeuchi, 1999 and Hall *et al.*, 2003):

- The physical (geomorphological, hydrological and hydraulic) processes involved in flooding.
- Flood control structures, such as drainage systems, storage reservoirs and flood defences.
- Economic, social and environmental assets that are impacted on by flooding and/or influence flooding processes.
- Organisations with a statutory responsibility for managing flood risk.
- Insurance companies providing cover for flood risk (thereby acting as a means of transferring or sharing flood risk).
- Other stakeholders effected by flooding or interventions taken to manage flood risk.

These sub-systems will contain a mixture of 'hardware' (plant, computers, instrumentation *etc.*), 'software' (databases, models *etc.*) and 'bioware' (people, the environment *etc.*) (Wymore, 1993).

Changes in flood risk

Intervention strategies can alter the behaviour of different parts of these sub-systems, some examples of possible human interventions are given in Table 2.1. Changes may also result from natural processes such as climate change or long term geomorphological evolution. Figure 2.4 shows how human and natural interventions can alter flood risk. It can be seen that enhancement or deterioration of the defence infrastructure act to increase or decrease the probability of inundation. Public education and improved flood warning will reduce the consequences of flooding. Uncontrolled development in the floodplain will clearly act to increase flood consequences, however, the probability of flooding may be increased due to the loss of drainage and alteration of floodplain properties.

Table 2.1 Examples of flood risk management interventions (Hall et al., 2003)

Intervention	Effect of action	Role
Development control in floodplains	Limit to construction of buildings and infrastructure in the flood plain, hence controlled increase in vulnerability	Planning authorities
Improving flood resistance of buildings	Reduced flood damage	Property developers and building owners
Increasing public awareness of temporary measures to reduce flood impact on building contents	More effective public action to reduce flood damage to building contents	Building occupants
Flood insurance	Redistribution of the cost of damage across the population and through time	Insurance companies
Increasing storage in catchments and reducing the rate of runoff (source control)	Reduced flood severity	Property and infrastructure developers, planning authorities and farmers.
Urban drainage	Reduced probability of flooding.	Water authorities, local authorities and highways authorities.
Flood defence • Planning • Design • Construction • Operation • Maintenance	Reduced probability of flooding (up to events that overtop the defences)	Environment Agency and local authorities.
Soft engineering *eg*. beach nourishment and vegetation management	Reduce vulnerability of defences	Environment Agency and local authorities
Real time flood forecasting and warning	Reduced flood impact (if followed by appropriate action by the public)	Environment Agency and Meteorological Office
Emergency repair of flood defences	Reduced probability of flooding	Environment Agency and emergency services
Evacuation of people in flood events	Reduced public safety and health impacts of flooding	Emergency services
Post-flood recovery and reconstruction	Reduced social, health and economic impacts of flooding	Local authorities and insurers

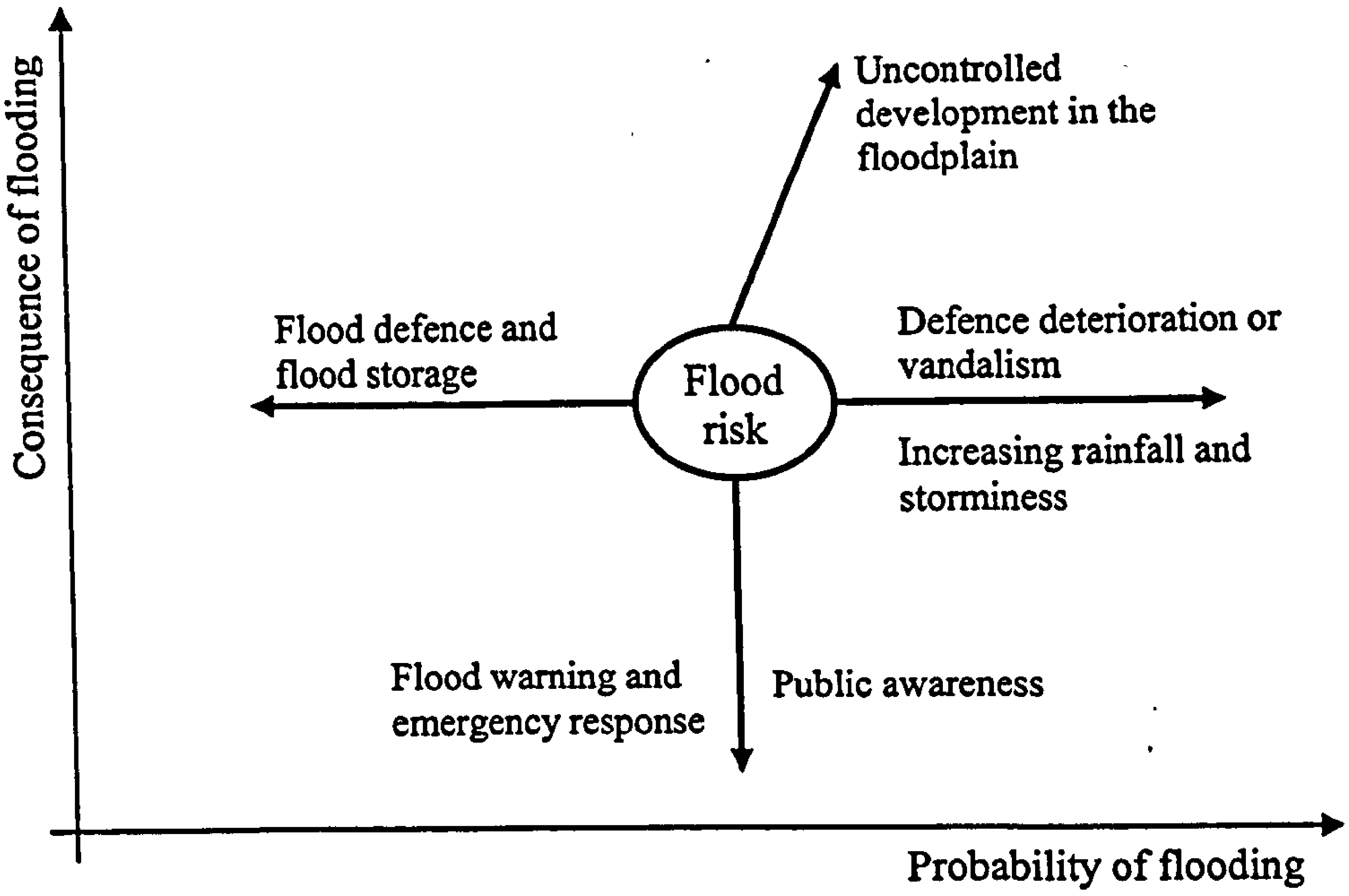

Figure 2.4 How human and natural interventions can change flood risk (Sayers et al., 2002)

2.7.3. Risk assessment methods and techniques

A risk assessment involves (Stewart and Melchers, 1997):

(1) identification of the hazards,

(2) identification and estimation of the consequences of these hazards,

(3) estimation of the likelihood of a hazard occurring,

(4) evaluation of the significance of the risks, and,

(5) identification and evaluation of the uncertainties.

Risk management encompasses the risk assessment process, how it is used within the decision-making process and also involves the mitigation of risks and uncertainties. DEFRA (2000b) recommend flood defence managers in England and Wales take a tiered approach to risk assessment, with qualitative approaches being recommended for shoreline and catchment management plans and where possible, quantitative methods being applied for the more detailed strategy plans.

Identification of hazards

Hazard identification identifies the sources of risk to the system. This process is usually undertaken by a team of experts applying one of a number of techniques to aid them. These techniques range from an initial brainstorming or Preliminary Hazard Analysis (PHA) (Stewart and Melchers, 1997) through more formalised techniques such as the Structured What-If Checklist (SWIFT) (HSE, 2001), Failure Modes and Effect Analysis (FMEA) (Stamatis, 1995), Failure Mode, Effect and Criticality Analysis (FMECA) (CIRIA, 2001) to specialised techniques for highly complex systems, such as Hazard and Operability (HAZOP) (Kletz, 1997).

Changes in flood risk

Intervention strategies can alter the behaviour of different parts of these sub-systems, some examples of possible human interventions are given in Table 2.1. Changes may also result from natural processes such as climate change or long term geomorphological evolution. Figure 2.4 shows how human and natural interventions can alter flood risk. It can be seen that enhancement or deterioration of the defence infrastructure act to increase or decrease the probability of inundation. Public education and improved flood warning will reduce the consequences of flooding. Uncontrolled development in the floodplain will clearly act to increase flood consequences, however, the probability of flooding may be increased due to the loss of drainage and alteration of floodplain properties.

Table 2.1 Examples of flood risk management interventions (Hall et al., 2003)

Intervention	Effect of action	Role
Development control in floodplains	Limit to construction of buildings and infrastructure in the flood plain, hence controlled increase in vulnerability	Planning authorities
Improving flood resistance of buildings	Reduced flood damage	Property developers and building owners
Increasing public awareness of temporary measures to reduce flood impact on building contents	More effective public action to reduce flood damage to building contents	Building occupants
Flood insurance	Redistribution of the cost of damage across the population and through time	Insurance companies
Increasing storage in catchments and reducing the rate of runoff (source control)	Reduced flood severity	Property and infrastructure developers, planning authorities and farmers
Urban drainage	Reduced probability of flooding.	Water authorities, local authorities and highways authorities.
Flood defence • Planning • Design • Construction • Operation • Maintenance	Reduced probability of flooding (up to events that overtop the defences)	Environment Agency and local authorities.
Soft engineering eg. beach nourishment and vegetation management	Reduce vulnerability of defences	Environment Agency and local authorities
Real time flood forecasting and warning	Reduced flood impact (if followed by appropriate action by the public)	Environment Agency and Meteorological Office
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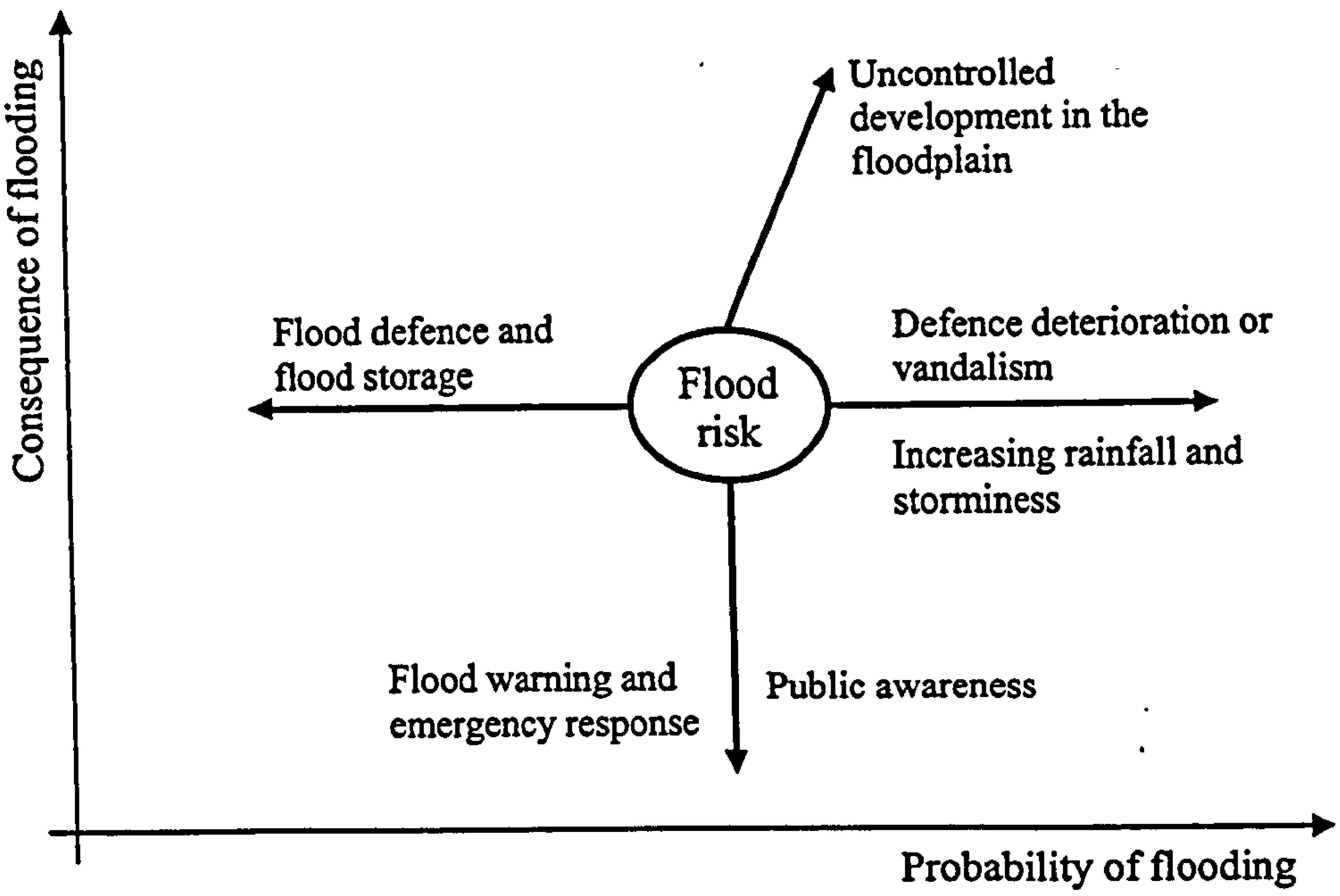

Figure 2.4 How human and natural interventions can change flood risk (Sayers et al., 2002)

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Indicative methods of risk assessment

Risk registers and matrices are frequently used to tabulate and record risks and document the decisions, for example steps taken to mitigate the risk. These are mostly associated with project risk assessment. An example of a risk register is shown in Table 2.2.

Table 2.2 Example of a risk register (DEFRA, 2000b)

Project No.	**Project Manager**					
Project Name						
Project Description						
Risks	Probability	Consequence	Assessment	Mitigation	Action By	Residual Risk

Frequently precise probabilities of occurrence of an event are not available, or the cost to obtain them would be inappropriate for the level of analysis of the risk assessment. Linguistic bands are therefore used to describe the probability, for example, ranging from 'frequent' through 'occasional' to 'improbable'. These can be correlated to numerical values if these are required for a quantitative analysis. The mitigation measure is designed to reduce the risk that has been identified (this could involve reducing the consequences or the probability of occurrence or a combination of the two). The residual risk remaining after this action is taken should be identified, and if necessary take further mitigating measures (CIRIA, 1996).

Risk matrices of likelihood and consequence can be a convenient and useful method of displaying this information (CIRIA, 1996), as shown in Table 2.3, where darker shades of grey are used to represent greater risks. This method is recommended by CIRIA (2001) for a simple strategic level risk assessment of infrastructure embankment condition in order to categorise the risk and identify actions to be taken.

Table 2.3 An example of a risk matrix, with darker shades representing a higher risk

			Consequences		
Likelihood	**Insignificant**	**Minor**	**Moderate**	**Major**	**Catastrophic**
Frequent					
Probable					
Occasional					
Remote					
Improbable					

Comparative qualitative methods, such as the risk matrix, are useful in that they allow different types of hazard to be compared on the same scale, require a relatively small amount of specialist skill and allow easy prioritisation of the risks. However, they require many judgements and one of their weaknesses is the inconsistency with which these are assessed. Hazards may also have many possible levels of consequence, for example, tripping on a banana skin may result in anything from a bruise to a broken back (Stewart and Melchers, 1997). The connectivity between different aspects of the system, for example how several small hazards occurring at the same time may have a consequence greater than the sum of their individual consequences, is not considered.

Some of these problems are addressed by Failure mode element and criticality analysis (FMECA). This prioritises the risk of failure associated with individual structural components. FMECA combines event trees with a risk register to produce a location, cause, indicator diagram (Figure 2.5) that can be used to rank each failure mode according to the combined likelihood of occurrence, consequence and confidence.

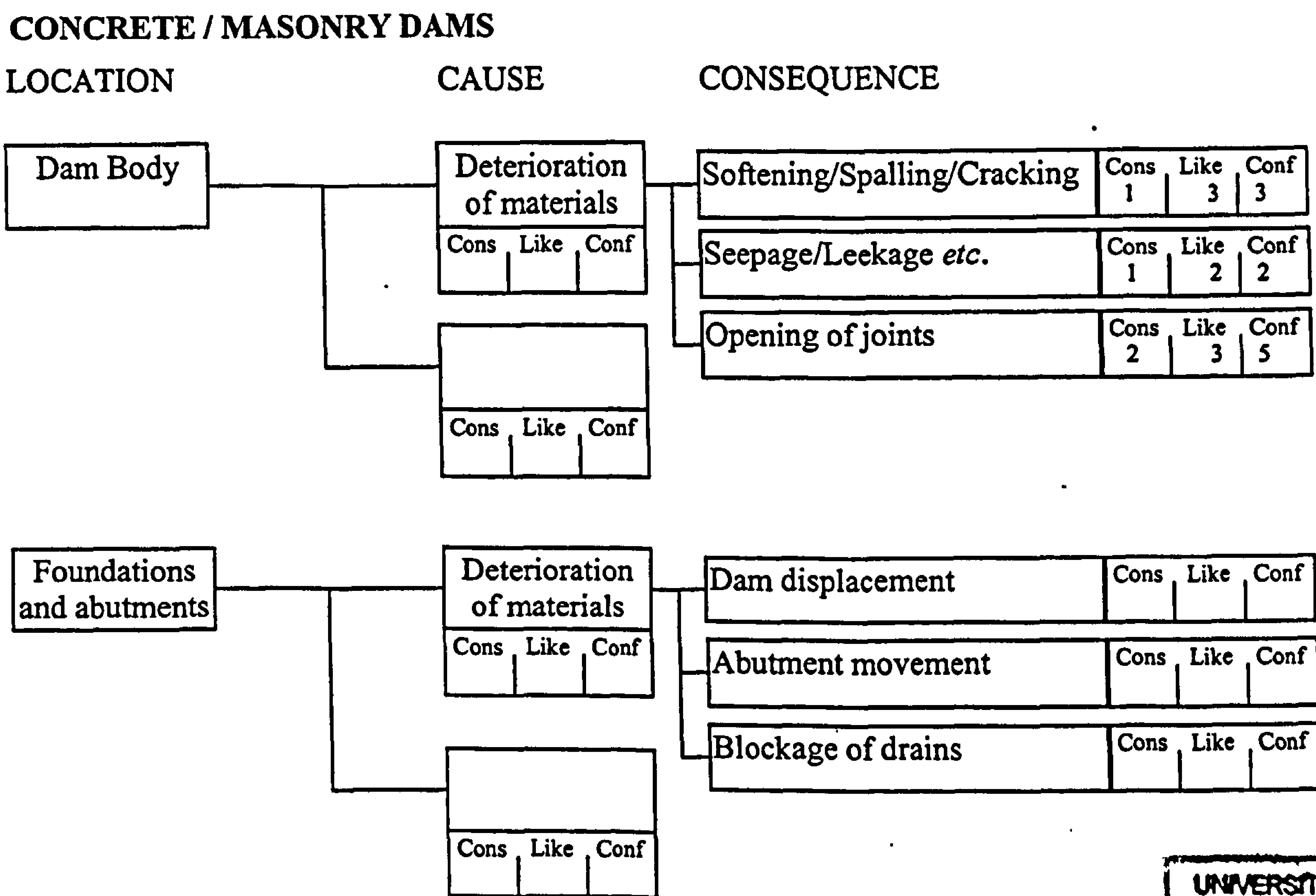

Figure 2.5 Part of a FMECA analysis of a concrete dam (Morris et al., 2000)

The criticality of an element is assessed by multiplying the likelihood, consequence and confidence score. This is multiplied by the impact to provide a measure of risk (Morris *et al.*, 2000). This offers a mechanism for considering risk in a transparent and auditable framework without the need for excessive probabilistic analysis. This has been tested within the UK dams industry using quantitative comparisons of criticality, confidence and consequences (CIRIA, 2000), similar methods have been adapted in British Columbia in order to perform a full probabilistic analysis of dam safety (Nielsen *et al.*, 1994).

Event trees are tools that describe the sequences of events from an initial event to a final outcome (HSE, 2001). For example, inadequate filter layers in a revetment can lead to a loss of fill beneath the revetment, resulting in a loss of strength of the revetment, resulting in premature damage of the revetment. The causes and effects can be modelled in a fault or event tree. An event tree is a logical representation of the events that may follow from an initiating event (*eg*. high water levels). An example of an event tree for a flood defence is shown in Figure 2.6.

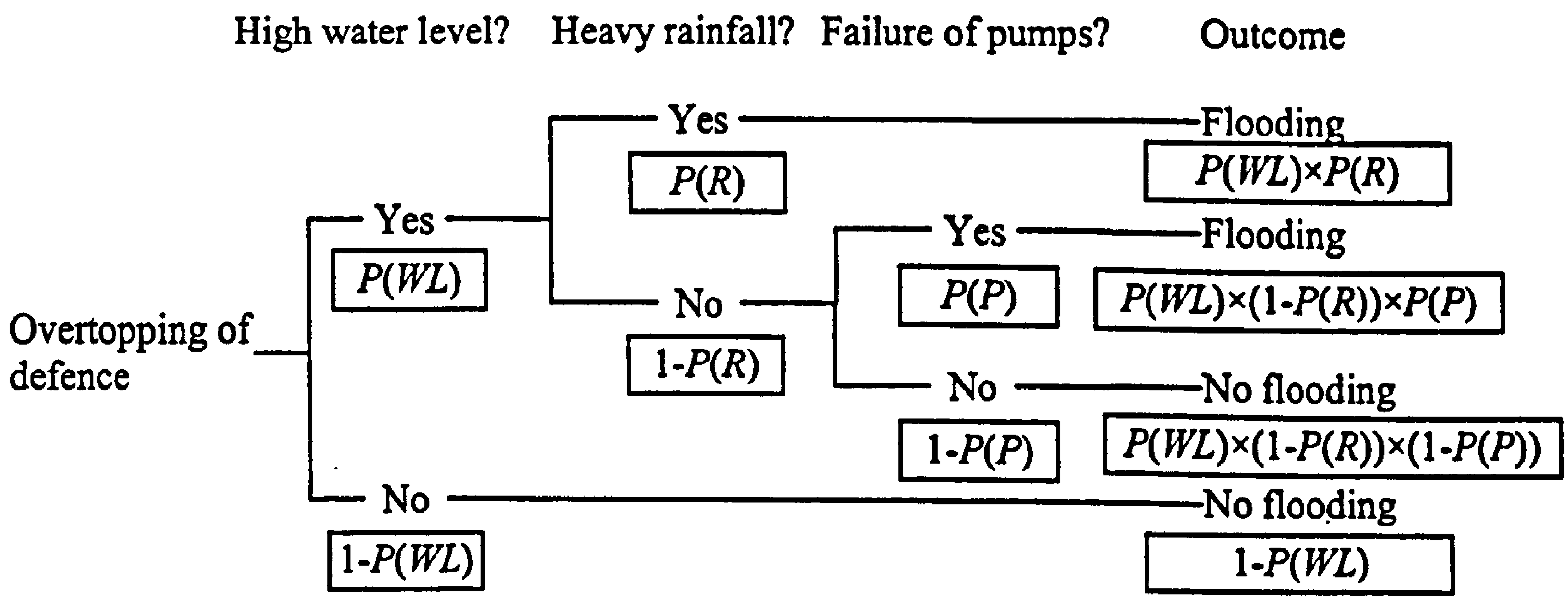

Figure 2.6 An example of an event tree

Fault trees often appear similar to event trees, but they are a logical diagram showing all the failure or partial failure mechanisms that contribute to the failure of a structure (Thomas and Hall, 1992) Figure 2.7 shows an example of a fault tree. The AND gate represents the assumption of independence whilst the OR gate represents mutual exclusivity.

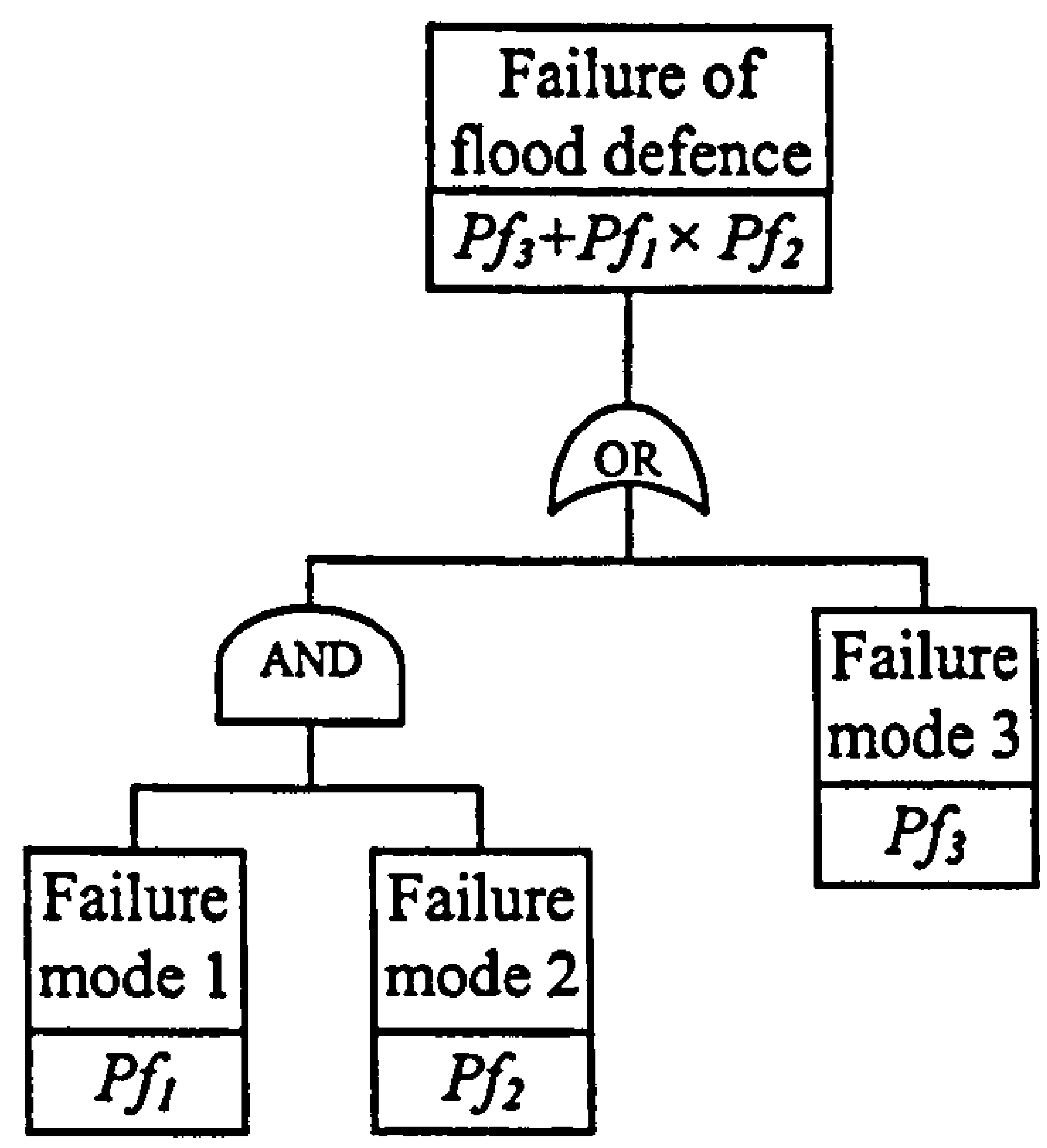

Figure 2.7 An example of a fault tree

Probabilistic methods

Many quantitative methods, such as fault trees, require the estimation of failure probabilities. Present guidance in England and Wales, described in Section 2.5 and Appendix C, requires the use of probabilistic discounting. Failure probabilities and degradation rates are assigned by experts. In the context of flooding, flood events are best described in terms of probability distributions to reflect the likely range of loads to which they are subjected. Uncertainties in defence response have also been represented using probability distributions (see for example Oumeraci *et al.*, 2001). The main steps to making a probabilistic assessment of flood risk involve:

(1) describing loads (*eg.* water level, marine storminess) probabilistically,

(2) describing defence system response probabilistically,

(3) modelling inundation, and,

(4) estimating the consequences of defence failure over a range of flood events.

Description of loads as joint probability distributions using methods such as those described by Hawkes *et al.* (2002) that consider joint loadings in the coastal environment are widely applied in current practice (CIRIA and CUR, 1991, CUR and TAW, 1990). River loads can be described probabilistically and this has been well established in current practice by the Flood Estimation Handbook (CEHW, 1999).

Extensive research (including CUR and TAW, 1990, Casciati, 1992, Lamberti, 1992, Reeve and Burgess, 1994, Wolff, 1997, Cooke *et al.*, 1997, Madrell *et al.*, 1998, Reeve, 1998, Martinelli *et al.*, 2000, Oumeraci *et al.*, 2001, Voortman *et al.*, 2002, Reeve, 2002) has studied the estimation of flood defence failure probabilities using reliability methods. However, their application to flood defence management in England and Wales has been limited (predominantly due to insufficient data). In the Netherlands where rather more data on flood defences is available, they have started to use the PC-RING software (Steenbergen, 2001) to perform a full probabilistic analysis for each of their ring dyke systems. This software estimates the failure probability of the dyke system using first order reliability methods (see Section 3.3.2 for a detailed explanation of these techniques) to estimate failure probabilities for a number of failure modes for each dyke section. The failure modes analysed are overtopping, piping, inner slope stability, outer slope stability and revetment failure. To perform the reliability analysis a complete set of information on loadings and defence property is required. PC-Ring assumes parameters are suitably expressed as either a point value or a probability distribution. Certain values, such as the variance of soil cohesion, are usually based on expert judgement, or a representative default value, rather than an investigative study of the soil (Harr, 1995). Whilst the software is specifically set up to assess the ring dyke systems that are so common in the Netherlands, the theory behind the reliability analysis is transferable. However, implementation of these methods in the UK would be limited by the lack of necessary information and the cost associated with obtaining it.

The USACE (1996) more commonly use expert judgement to construct relationships between water height and failure probability rather than more explicit approach proposed by Wolff (1997). Wolff's approach calculates the failure probability of failure modes that can be described by limit state functions using First Order Second Moment (FOSM) methods (these are described in detail in Chapter 3). However, Wolff acknowledges that limit states are not well developed for some functions, if at all, and introduced an additional 'judgement' limit state function that elicits expert judgment to account for other items not explicitly modelled. This 'judgement function' is analysed in the same manner as a traditional limit state function using FOSM methods. A total failure probability for a structure is calculated by making the conservative assumption that the failure modes are independent. The most recent guidance on geotechnical reliability (USACE, 1999) recommends the use of fault trees populated with probabilities obtained either from reliability calculations or expert judgement to calculate levée failure probabilities. HEC software (USACE, 1998) is used to support the calculations in the guidance and uses several packages to perform the hydrological, hydrodynamic, reliability and economic analyses to calculate flood risks. This method clearly offers the benefit of considering the key processes in assessing flood risk, but simplifications have been made as the system is not fully integrated; for example, the effect of failure of upstream defences on downstream loads.

Reliability, uncertainty and the theory behind probabilistic risk assessment are elaborated in Chapter 3.

Additional methods

There are other tools a decision-maker can use to help support decisions in a risk framework and some of these have been reviewed by HR Wallingford (2002). Ultimately the choice of method should reflect the decision being supported, for example a radial uncertainty chart provides a means to graphically assess the importance of different uncertainties and would be used when identification of the largest uncertainties was a key factor influencing a decision.

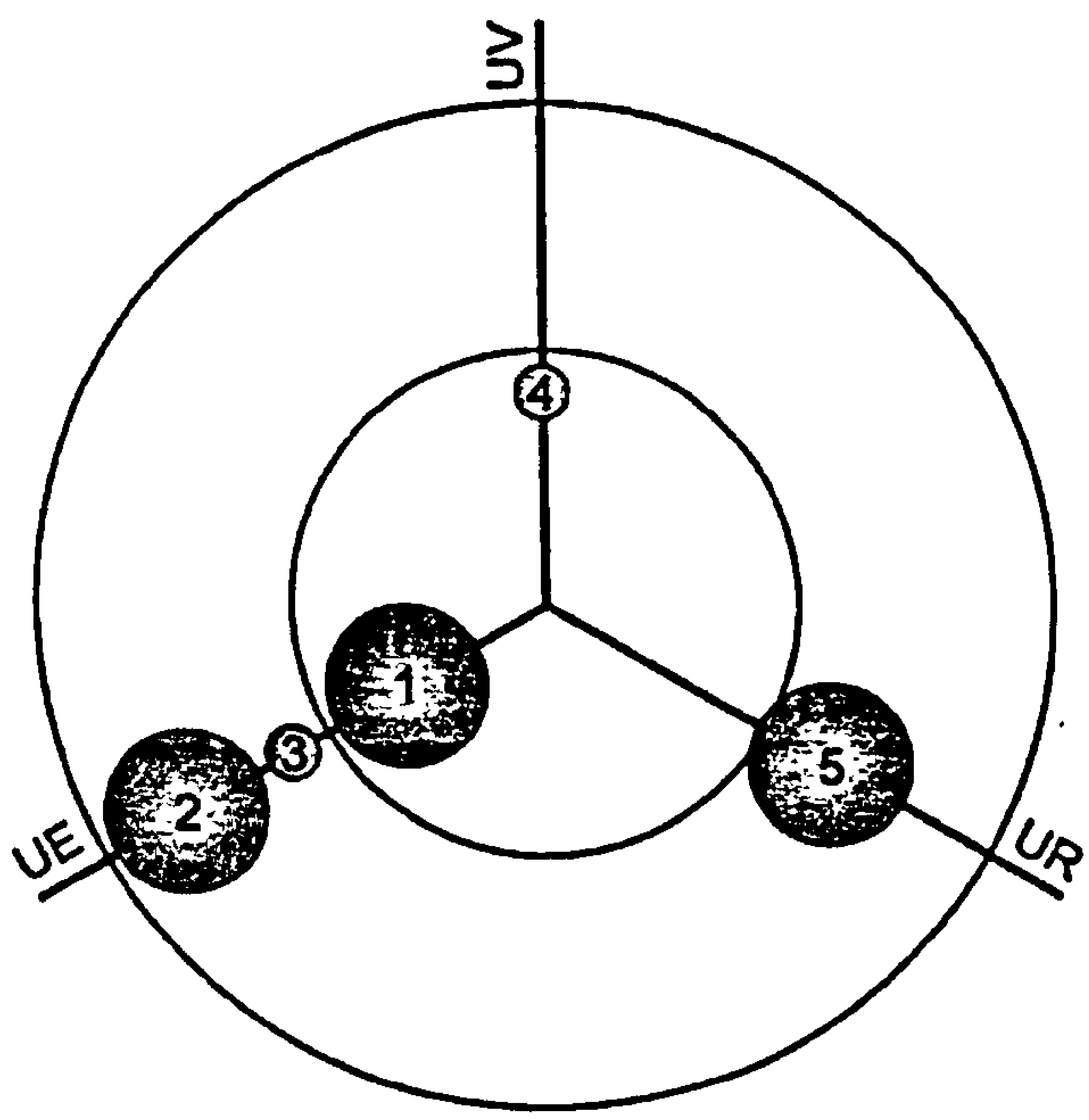

Figure 2.8 A radial uncertainty chart (HR Wallingford, 2002)

Figure 2.8 shows an example of how a radial uncertainty chart for a beach nourishment project may look. Along each line are uncertainties in the future environment (UE), uncertainties in values, costs or quantities (UV) and related decisions (often made by third parties) that may influence the long term outcome of the project (UR). The circles in Figure 2.8 represent the uncertainty in sea level rise (1), uncertainty in global circulation models (2), uncertainty in future CO_2 levels (3), uncertainty in project costs (4) and uncertainty in future legislation related to offshore dredging (5).

2.7.4. Risk as a decision-making tool

The move towards integrated flood risk assessment brings many advantages to flood defence management. These advantages are that it:

- allows consideration of the entire flood defence system and the interconnectivity of its components,
- incorporates uncertainties associated with the assessment of system behaviour,
- offers an insight into the sensitivity of the flood risk to these uncertainties,
- explicitly considers the costs of improving the system and impacts of system failure,
- is transparent and auditable, and,
- provides a logical and rational basis for development of flood management policy and resources and monitoring the performance of flood mitigation activities.

Because of this, risk assessments can be used to support a broad range of decisions relating to flood and coastal defence systems, these include:

- strategic planning at a national, catchment and coastal cell level,
- planning of flood warning and emergency response,
- regulation of development,
- prioritisation and justification of capital schemes,
- prioritisation and justification of maintenance,
- prioritisation and optimisation of monitoring strategies,
- identification and optimisation of data collection strategies, and,
- scheme design and optimisation of both hard and soft defences.

In general, decision-making involves selecting a preferred option of intervention for the system by considering the predicted performance of the system in terms of meeting a set of performance objectives (DEFRA, 2000b). For flood and coastal defence managers, these are usually indicators of economical, social and environmental performance. Maximising the value associated with an option, by minimising the environmental, social and economic risks, provides the decision-maker with a logical manner on which to base their decision (Hall, 2000). Areas of higher risk can be targeted for investment and the benefits of different interventions can be explored and justified in an auditable and transparent manner. A good risk assessment will also identify and evaluate the uncertainties allowing a decision-maker to account for these in the decision and mitigate them where possible. This is especially important in flood and coastal defence where there is often considerable uncertainty in determining the probability and consequences of flood events (HR Wallingford, 2002). The benefits of data acquisition schemes and their subsequent effect on uncertainty can be explored within a risk-based framework. Quantitative risk assessments can be used to perform the economic optimisation of structural design. This has been formalised by Sorensen *et al.* (1994), Voortman *et al.* (1998) and Tung (2000).

$$\min C(\underline{z}) = C_0 + C_I(\underline{z}) + \sum_{n=1}^{N} \frac{C_f . P_f(\underline{z})}{(1+r'-g)^n} + \frac{C_{main}}{(1+r')^n} \tag{2.4}$$

where $C(\underline{z})$ is the total cost as a function of the vector of design variables $\underline{z}$, C_0 represents costs not dependent on the design variable such as initial studies, $C_I(\underline{z})$ construction costs as a function of design variables (including time), C_f damage costs from failure, C_{main} maintenance costs, P_f probability of failure, r' is the net interest rate, g is the yearly rate of economic growth and N is the structure's lifetime. Economic optimisation of soft defences, such as beach nourishment schemes, has been demonstrated through integration of sediment transport models, simulations of nourishment strategies and probabilistic discounting of costs and consequences by Dawson and Flory (1999), van Noortwijk and Peerbolte (2000) and Johnson and Hall (2002).

Assessment of risk brings many advantages to flood defence management as it provides a useful and logical basis for supporting a wide range of decisions. However, it should be noted that risk-based management is not without potential pitfalls that need to be avoided (Hall, 2000). These range from the practicalities of obtaining and using evidence in a risk assessment to more philosophical issues related to modelling and decision-making and are expanded upon in the following sections.

Risk communication and perception

How risk is communicated and perceived by others is a potential problem with disseminating probabilistic risk assessments (Royal Society, 1993). Numerical risk assessments can be accepted as fact rather than the partially subjective evaluations they are (Bedford and Cooke, 2001). Engineers in the UK also have difficulty in communicating even relatively basic risk concepts to members of the general public (Hall *et al.*, 1998).

People's reaction to risks will depend on their values (organisational, cultural *etc.*) and objectives, for example some people will value economic risks more highly than environmental risks. Public opinion can also be heavily influenced by the media. Less frequent, but higher consequence events, for example a train crash, will often receive more attention than more frequent, lower consequence events, such as a car crash (Royal Society, 1983). Risks may be voluntarily accepted, for example dangerous activities such as parachuting. Westerners are usually prepared to accept risks between 100 and 1000 times greater if they are voluntary (Starr, 1969). Flooding is rarely perceived to be a voluntary risk. Whilst the UK government favours taking a risk neutral (as opposed to risk averse or risk seeking) perspective to investment decisions (HM Treasury, 1997) in order that over a long time the net value for the tax payer is maximised, the attitudes of the public towards serious flood events should be considered and it may often be necessary to adopt a risk averse (accepting a lower risk than the expected risk) decision-making strategy that constrains the risk of extreme events.

Beck (1992) and DETR *et al.* (2000) warn that risk assessment needs to be a transparent and inclusive process to ensure that maximum support is gained from the public and decision-makers. Therefore to ensure that efforts to create ever more sophisticated and improved techniques for risk assessment are accepted into the decision-making process, the risks and the assessment methodologies must be appropriately communicated.

Modelling

A quantitative risk assessment by its very nature requires modelling. The sophistication of the models ranges from a simple limit state description of failure to a continuous simulation of an entire system. A model is by definition an abstraction of reality and will only ever be an approximation of the processes governing system behaviour. This approximation is due to (Davis and Blockley, 1996):

- that which cannot be foreseen, perhaps because the phenomena are previously unknown, or the interconnectivity between phenomena is not understood, and,
- that which is foreseeable but is:
  (a) ignored by mistake,
  (b) ignored by choice, perhaps because the process is considered unimportant, or,
  (c) ignored because of a lack of resources, perhaps because the process is too complex to model or necessary data is too costly to obtain.

There are often limits to the predictability of some processes in the long term. For example, De Vriend (1991) argues that long term trends in coastal evolution are "a weak residual of a very 'noisy' signal of short term variability".

Measuring modelling uncertainties involves devoting a large amount of effort and computer resources. The integration of probabilistic methods into these models adds a further order of complexity (Hall, 2000). For these reasons, it is likely that this is why the majority of modellers use models that are 'fit-for-purpose' and it is relatively rare for modelling uncertainties to be accounted for (Beven, 2002). However, even if model uncertainties are not explicitly measured a decision-maker needs to be aware (if only in a qualitative sense) of the limitations of the models as ultimately, the risk assessment reflects their completeness and dependability and this should be considered in subsequent decision-making.

Expert judgement

There is much evidence that can be used to assess flood risk. Hall (1999) identified 117 data collection and analysis processes for a single managed set back project on the East Coast of the UK. The amount and type of uncertainty will vary according to the type of evidence and collection processes. As identified in Sections 2.5, 2.6 and 2.8, expert judgement plays an important role in decision-making in England and Wales. Ideally probabilities used in risk assessment are constructed from quantitative or statistical analysis, although the choice of method used for the analysis and the applicability of the data used are in themselves subjective judgements. Expert judgements are naturally prone to bias (Kahneman *et al.,* 1982 and Cooke, 1991) and to minimise this bias, the decision-maker should (Hall, 2000, DEFRA, 2000b):

- define the event to which the judgement is being attached precisely,
- structure the problem logically so initiating events can be linked to consequences,
- check the judgement for inconsistencies by logically structuring the elicitation and obtaining other judgements that can then be subjected to peer review and criticism,
- use available quantitative information to inform expert judgement,
- document and structure the evidence and process by which the judgement is elicited, and,
- assign 'most likely' or 'best' and 'worst' estimates rather than point estimates.

Expert judgements are an important source of evidence of system behaviour and should not be ignored. However, these judgements, be they in the form of evidence of performance, or judgements about the suitability of a particular model should be acknowledged and made transparent in the decision-making process. The uncertainty associated with these judgements should be acknowledged and evident in the risk assessment, this issue is discussed below.

Capturing uncertainty

Lindley (1971) argues that probability can be used to capture all forms of uncertainty. There are many situations in which probabilistic handling of uncertainty will be useful to the flood defence manager. For example, river flow cannot be predicted with certainty, but given historical flow records it is possible to construct a probability distribution of flow rates. However, in many decision-making situations information is incomplete or in the form of expert opinions expressed in vague linguistic terms. The uncertainty associated with this data needs to be appropriately represented in the risk assessment. Current approaches to quantitative risk assessment in England and Wales require judgements on defence strength to be expressed as subjective probabilities of failure. However, this sort of discrete representation of failure probability is inappropriate for capturing a judgement that is characterised by vague, incomplete or ambiguous information. This type of uncertainty, or fuzziness, may be represented as interval bounds or fuzzy sets. These provide a description of the possible space in which the actual value is expected to lie, without endeavouring to attach probabilities of the likelihood of the value. Fuzzy descriptions of parameters have been proven useful in capturing uncertainty in structural condition (Yao, 1997) and geotechnical conditions (Dodagoudar and Venkatachalam, 2000). Where uncertainty is less amenable to representation as a probability distribution it should be expressed in an appropriate syntax in the risk assessment.

Decision-making

The economics of decisions are compared by DEFRA (2000a, 2002) using several techniques. The shortcomings of one of the simplest tools, a benefit-cost analysis are readily acknowledged (Hall *et al.*, 2003). Discounting is used to evaluate the whole life cost and benefit economics of schemes, however its shortcomings for valuing sustainability are recognised by Adams (1995). This has been recognised at a national level (DETR, 1999b) where economics is considered alongside the environment, health and other social indicators as evidence of sustainable development. The short fallings of economic tools needs to be considered in decision-making.

Traditionally the justification of intervention schemes that involve construction or maintenance of defences to reduce flood risk has focused on measuring the benefits in terms reducing economic losses (DEFRA, 1993). Critical reviews (Bye and Horner, 1998, ICE, 2001) have resulted in a more explicitly multi-attribute approach (HM Treasury, 1999) that also considers social and environmental indicators (DEFRA, 2002). However, care must be taken with such approaches. These approaches are usually applied at the earlier stages of project management to reduce the number of schemes before a detailed appraisal. Multi-attribute methods do not currently take into account the uncertainty associated with the scoring system used which is more likely to be prevalent at an earlier stage in project appraisal. There are probabilistic ways of handling this uncertainty (Keeney and Raiffa, 1976) but Hall (2000) recommends an interval approach. Whilst this will not necessarily enable a single preferred option to be identified, it is usually more appropriate to identify a handful of options for further study.

Decision-makers should be aware that whilst a risk assessment provides an 'optimum' solution, this is based only on those effects considered in the analysis and is therefore vulnerable to events not included. These effects can be mitigated by good risk management that includes monitoring, managing vulnerability, reducing response time, reducing mitigation costs and remaining flexible (Collingridge, 1980). This compliments the observational method that was introduced to geotechnical engineering (Peck, 1969 and CIRIA, 1999). This method suggests that where there is a high degree of uncertainty it is better to prepare for the most likely conditions and be ready to adapt the design based on monitoring. This approach is most useful if the decision-maker is able to adapt their decisions and update their models as more information becomes available, and often forms the basis of beach nourishment techniques (Hall, 2000). It may be necessary to weigh the apparent loss of flexibility in adopting the 'optimum' decision against the loss in flexibility. Robustness will usually be favoured in conditions of high uncertainty. This is demonstrated in Figure 2.9 where scheme B has the highest risk, and based only on this information scheme A would appear preferable. However, the uncertainty associated with scheme A is much greater. A decision-maker has to therefore weigh up the advantages of a greater expected reduction in risk against having a more robust scheme.

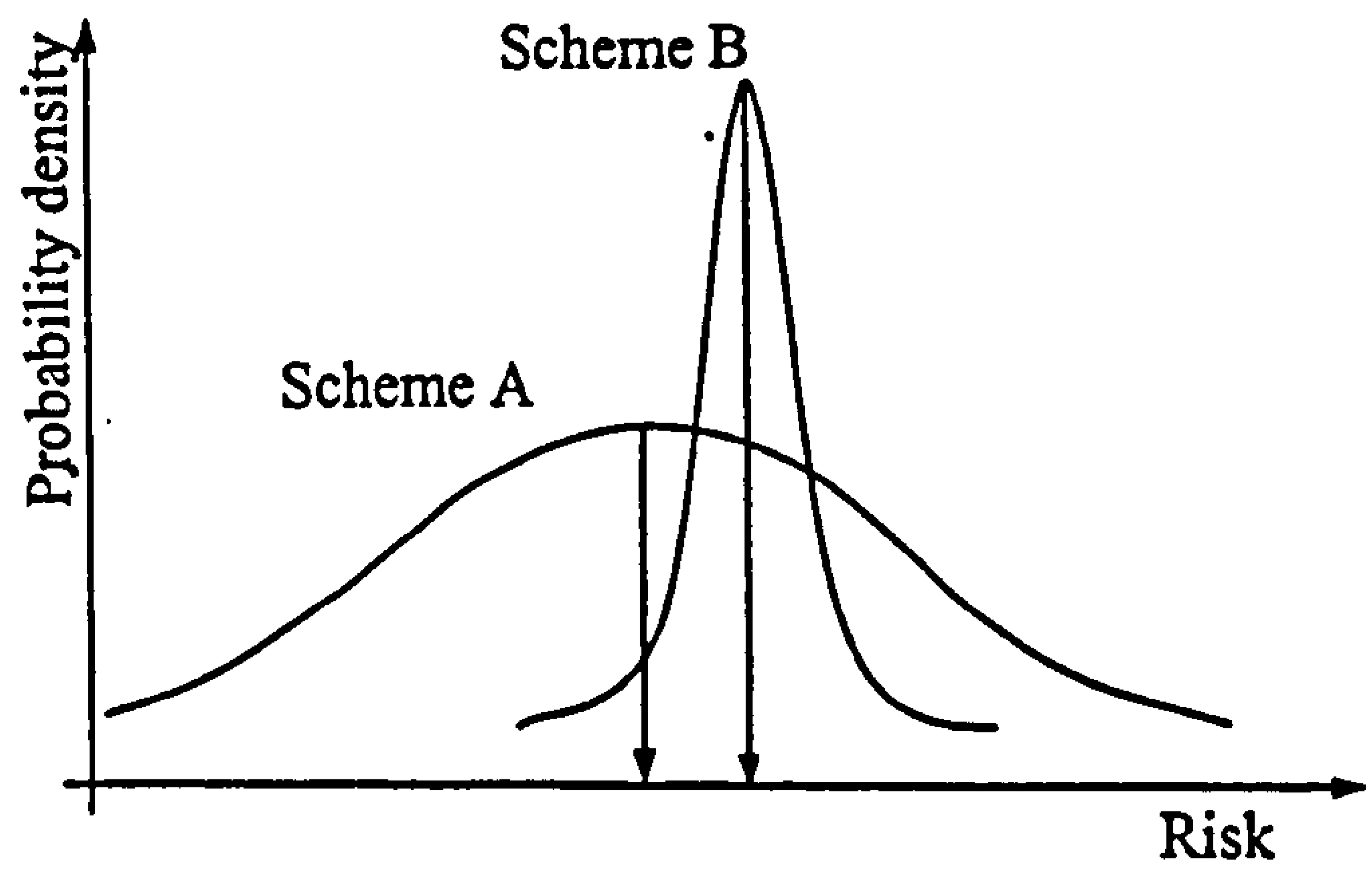

Figure 2.9 Uncertainty in the decision-making process

2.8. CURRENT PRACTICE IN FLOOD DEFENCE MANAGEMENT

The previous sections have provided a review of the techniques and guidance available to flood defence managers in England and Wales. An insight into less formalised procedures and actual practice which was obtained through:

- visits to three Environment Agency offices,
- informal interviews with Environment Agency staff,
- informal discussion (in the form of interactive seminars) with experts and professionals involved in flood defence and infrastructure management, and,
- site visits with EA officers and separately with sub-contracted engineering consultant that included condition assessment inspections of flood defence works.

The findings of this study are summarised below.

Application of guidance

The FDMM is only applied to a limited extent. This uptake varies considerably amongst the regional and area offices. Data logging in the FDMS is also inconsistent, with some offices being able to provide records going back further than a decade and others with only recent visual inspections. A number of reasons were identified for this limited uptake:

(1) The FDMM superseded the previous procedures of the individual regions. Whilst national consistency was recognised as being desirable, it was felt that the FDMM was inferior to the procedures already in place.

(2) The procedures laid out in the FDMM appear laborious and time consuming to decision-makers and the value which they provide is not obvious.

(3) Information in legacy databases could often not be stored within the FDMS (due to insufficient database fields).

(4) The FDMS is not a well designed, user friendly piece of software. A front-end has since been created to enable compatibility with more accessible databases.

(5) Capital projects that are applying for DEFRA grant aid (and frequently those not applying for grant aid) are required to follow different guidance.

In April 2002, the NFCDD took over from the FDMS as the EA's database. The procedures within the FDMM are currently being reviewed.

Limited application of the FDMM, the ongoing review of asset management procedures and the recognition by flood defence managers of the benefits available from risk-based management has resulted in interim approaches being adopted by different EA regional offices. These vary according to the resolution at with the risk is measured, being at either the reach or individual defence level. Limited data has required that these methods are based on a comparison of defence condition and a crude measure of the consequences of flooding based on the land use along the reach. These qualitative risk-based approaches are used to support prioritisation of maintenance and inspections. Aside from recent estimates of an expected annual damage for England and Wales (Halcrow, 2001), flood risk tends not to be quantified.

Other guidance such as DEFRA's FCDPAG series and SMPs (and in the near future, CFMPs) are more rigorously applied than the FDMM. The greater flexibility in these guidelines allows the latest technological advancements to be applied. Aside from meeting legal requirements (such as, for example, satisfying the European Water Framework Directive (EU, 2000) capital projects are required to follow these more stringent guidelines if they are to apply for DEFRA grant aid. The scale and amount of funds being invested in capital projects will mean a detailed analysis is often more appropriate.

Data gathering and analysis

In the same manner as the degree of FDMM implementation varies, so do the EA's data gathering activities. Some regions commission full and detailed surveys of their assets similar to those suggested by the NRA (1993), others record just a visual inspection of asset condition (described in Section 2.6.1). The limited nature of this assessment means there is a large amount of uncertainty associated with this information. Its use as an indicator of the performance of a flood defence is limited and should therefore not be the sole piece of evidence on which investment decisions are based.

The value of data collection activities is not always recognised. Data is often collected based on an historical precedence rather than within a framework that weighs the use of the data against its cost. Regular monitoring of defence crest levels, one of the most important pieces of evidence needed to assess the performance of a flood defence (NRA, 1993, TAW, 1999b), does not take place nationally across England and Wales. Whilst measurement of crest levels is important, their use is limited without loading information.

The resolution and level of analysis of data is not always appropriate to the decision it is being used to inform. Frequently information content is lost through discretisation of detailed datasets. For example, land use bands categorise the floodplain behind a reach according to likely damages. They are often obtained from surveys that require precise evaluation of the assets in the floodplain. This evaluation is then discretised crudely into five bands, which could be as accurately and more cheaply estimated from the study of an Ordnance Survey map. This band and not the results from the detailed survey are used to support investment decisions.

Information collected by one initiative within the EA is not always communicated to other initiatives or departments. Data collected by local authorities has often been unavailable or difficult to obtain. The advent of the NFCDD is improving data communication, but this issue has also been recognised by DEFRA and Environment Agency (2001). However, even when data is adequately communicated the sheer volume means that without the benefit of the types of methods proposed in this thesis, decision-makers will struggle to maximise its usage.

Decision-making

The FDMM provides a decision-making methodology that has been automated to some extent within the FDMS. Although some aspects of the methodology can be considered to be quasi-risk-based in that some measure of consequence and proneness to defence failure are measured the usefulness of these methods is limited as they:

- are difficult to follow and frequently introduce unexplained coefficients, weightings and formulae,
- do not consider uncertainty,
- do not analyse data at a level appropriate to the scale of the decision being supported, and,
- do not allow comparison of different intervention options (*eg.* repairs, maintenance and capital projects).

As a result of this, the application of the FDMM to support decision-making is resisted by flood defence managers.

Flood Defence Officers (FDOs) are responsible for asset inspections and identifying the need for intervention works. They usually have a very good knowledge of their area of responsibility and the behaviour and condition of the assets within it. An understanding of an asset's behaviour is crucial in its management (NRA, 1993) and the experience of many of the FDOs means they have an overview of the performance of the system through time instead of the snapshot provided by an assessment using the Condition Assessment Manual. This knowledge and experience ensures they play a key role in identifying areas of concern and recommending action to be taken.

Boland *et al.* (1990) suggest that individuals are engaged in cycles of decision-making in their own domain. This corresponds to what was observed in the Environment Agency where decision-making by flood defence managers is much more likely to be a result of negotiation at parallel levels within the organisation. For example, prioritising maintenance works is dominated by the results of negotiation between managers and a number of FDOs working in different parts of the flood defence system rather than using the FDMM. The budget would be decided by negotiation between area managers, regional managers and the Regional or Local Flood Defence Committees. This is shown in Figure 2.10 that provides an overview of some of the flood defence infrastructure decision-making processes.

This suggests that there is a substantial reliance on the experience of the FDOs, a result of this is that a recommendation to invest money in a defence is made by just one person with little or no transparency or auditability in their decision-making process. It is only relatively recently that information on asset condition has been recorded centrally and in a nationally consistent manner and so there is potential for much of this 'corporate memory' to be lost when an FDO leaves or retires. To compound this loss, in-house expertise has been reduced due to down-sizing and out-sourcing of many technical services.

Large projects frequently involve many organisations (ranging from engineering consultancies and government organisations to charities and environmental bodies) and for larger projects a public consultation stage may be required. Aside from being required to make more detailed studies when applying for DEFRA funding, the involvement of the public or other organisations requires that information is externalised and decisions are made more transparent and auditable.

Funding

Funding of flood defences is obtained from many authorities. The majority of funding comes from a levy on council tax paid via local councils. The second most important source of funding is capital scheme grants from DEFRA (which range from 15-85% of project cost). Venables (1998) identifies lottery funds, EU funds and private partnerships as being other funding sources.

Because funding allocation is reviewed annually it becomes difficult to make long term budget plans. Though ten year plans are produced, in truth they are likely to become very unreliable after the third year. This is due partly to the uncertainty in predicting the future behaviour of the flood defence system and also because the council levies have to be re-negotiated on an annual basis. This ensures it is difficult to make long term plans, which is contrary to the aims of making management and strategy plans. Whilst there are good reasons for DEFRA to provide grant sizes that vary depending on the project and region, the size of grant can greatly influence the type of project. It can make more economic sense within a high-grant region to initiate a capital project, but without the grant the overall cost of a maintenance project would be less. This discourages a strategic approach to management. A recent funding review (DEFRA, 2002b) concurs with this and has recommended that the EA be given a block grant to reduce the uncertainty of the annual budget.

The nature of flooding also serves to undermine investment in defences because the return period of severe storm events is greater than the lifetime of the administrations that make the investment decisions. Periods of little significant flooding often results in complacency. Fund managers in central and local government will be pressurised to re-prioritise funding allocation to a higher profile cause. In addition to this, apathy from the general public results in increased losses at the next flood event due to a lack of preparedness. After a large flood event, funding for flood defence managers will be increased and the cycle continues. This has also been recognised by the ICE (2001) in a recent review of river management.

2.9. ISSUES IDENTIFIED AND RECOMMENDATIONS

Previous sections of this chapter have described the structure of flood defence management and the present methods used make flood defence investment decisions in England and Wales. It is clear from this review there are a number of aspects of flood defence management that can be improved. Three key areas have been identified.

(1) Risk assessment.

(2) Condition characterisation.

(3) Decision-support.

This section identifies the major weaknesses in the present state of the art of these areas and makes recommendations that the remainder of this thesis seeks to address.

2.9.1. Quantitative flood risk assessment

Section 2.7.4 identified the benefits to be gained from using a risk assessment to support decisions as long as the potential weaknesses are recognised and addressed where necessary. A flood risk assessment provides a key indicator of the performance of a flood defence system and is required to support the appraisal of policy options, allocation of resources and monitoring performance of investment in flood management. Flood risk management decisions take place at a number of levels, ranging from national policy decisions through planning in catchments and coastal cells and localised scheme design and operational decisions. Both government (DEFRA, 1999) and industry (ICE, 2001) have identified the need for improved management of flood risk.

FCDPAG4 (DEFRA, 2000b) provides guidance on risk and uncertainty issues, however, this guidance was written with the intention of providing support for the management of capital schemes. The guidance provides support for a broad range of aspects of risk assessment and management and offers only limited guidance for the assessment of the risk of flooding due to defence overtopping or breaching.

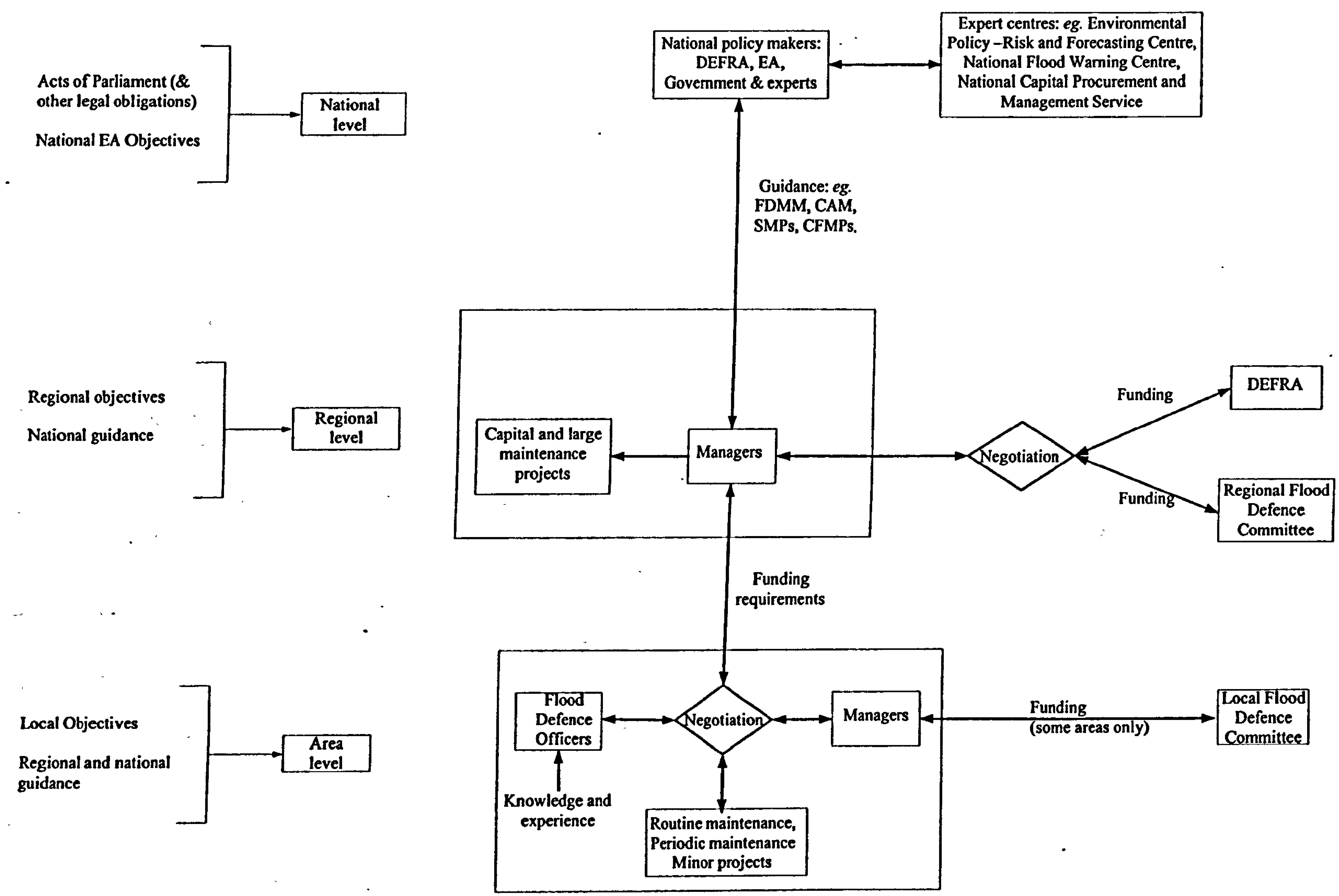

Figure 2.10 Overview of some of the infrastructure decision-making processes within the Environment Agency. The organisation can usefully be considered to have a hierarchical nature.

The use of risk assessment to support operations and maintenance decisions is currently limited. The Standard of Service (SoS) measure used in the FDMM provides an indicator of performance and information about expected annual damages. Decisions can be made based on a comparison of the current SoS and a target value. The use of the SoS methodology does provide a rudimentary risk-based framework as it compares a measure of likelihood and consequence. However, defence behaviour is not considered in the SoS meaning an important part of the flood defence system is neglected in the estimation of expected annual damages. Even if defence performance was considered, the SoS will not necessarily provide the optimal economic solution because of the limited consideration of costs and residual risk and the coarse discretisations involved in the calculations. Locally introduced risk-based methods provide a qualitative approach to the prioritisation of maintenance and inspection frequency. However, this type of approach is limited because:

- risk is only assessed in a comparative manner,
- uncertainties are not considered,
- the likelihood of flooding is assumed to be a function of only the defence condition,
- changes to the system condition, such as the increase in certainty from regular inspections or the increase in performance resulting from maintenance can not be measured in terms of risk,
- costs associated with the intervention strategies can not be assessed against the reduction in risk or uncertainty,
- qualitative measures of risk from a limited analysis of the consequences and likelihood of flooding are only suitable to minor investment decisions.

Recent guidance (DEFRA, 2001b and DEFRA, 2001c) has encouraged flood defence managers to work in a more strategic manner. This encourages flood defence decisions only to be made after considering their influence on other parts of the system, for example increased erosion down drift of a sea wall. Likewise, flood risk should be evaluated by considering the behaviour of the system. The interaction between defences, water levels and the floodplain all need to be considered when making an assessment of flood risk.

Quantitative risk assessment methodologies have been introduced in the USA and the Netherlands. Naturally, these have been tailored to suit the specific needs of the country or individual situations. These approaches are inappropriate for application in England and Wales because of the differences in organisational structure, the amounts of funding that are allocated, the availability of data and the criticality of flood defences as part of the national infrastructure. However, there is clearly a need for methods capable of a more explicit consideration and evaluation of risk to support flood defence investment decisions.

Improving the quality and comprehensiveness of knowledge used is one of the most useful ways of improving the risk assessment (National Research Council, 1992). However, the amount of resource, in terms of data acquisition and analysis committed to a risk assessment should reflect the nature of the decision(s) that the assessment seeks to inform (De Looff and Van Der Meer, 1998, Wang and Xiang, 2002, Sayers *et al.*, 2002). A tiered approach to systems risk assessment, building on the work of Meadowcroft *et al.* (1995) is proposed. The key recommendations for a risk assessment are summarised below.

- A tiered assessment that employs progressively more sophisticated analysis to support a range of decisions that vary from national funding allocation to maintenance prioritisation and individual scheme design.
- Quantification of flood risk using probabilistic methods.
- A broad definition of the flood defence system.
- An explicit consideration of the dependency and interconnectivity between elements of the flood defence system and the failure modes of these defence elements.
- Quantification of the uncertainties in the risk assessment.
- Information on loadings, defence performance and consequences should be separated until the final stage of the assessment in order to identify the sources of uncertainty.

A method that satisfies these requirements is described in Chapter 4.

2.9.2. Condition characterisation

A condition characterisation should provide a description of the structure's proneness to failure, enabling flood defence asset managers to make more informed decisions about maintenance, monitoring and replacement strategies. In England and Wales, the current condition characterisation philosophy is to consider the deterioration of the defence rather than directly analysing the defence's capacity to withstand various loadings as is the case in the Netherlands. Although grading defences on a scale of 1 to 5 is useful to an extent, it provides rather limited information about the defence's proneness to failure and its performance. Whilst the grading is based implicitly on the behaviour of the defence, it falls short of the explicit consideration of failure modes in the USACE methodology. Both the potential performance and degradation of the defence (and therefore the resulting loss in performance) needs to be addressed when making a condition characterisation. FCDPAG3 (DEFRA, 2000a) and FCDPAG4 (DEFRA, 2000b) do require the use and estimation of defence breach probabilities for use in the appraisal of capital projects. These probabilities are usually derived from expert judgement. The probabilities are point estimates and do not capture the uncertainty associated with the analysis or the fact that the failure probability is conditional on loading. FCDPAG4 also suggests how these failure probabilities may change over time to enable long term risks to be estimated, but this change is based on probability deterioration functions again, assigned by experts rather than model outputs or rates of deterioration of parameters that influence defence strength.

A condition characterisation should therefore provide a representation of defence performance over a complete range of loads. Uncertainty associated with this performance should be kept separate from uncertainties in loading. A condition characterisation methodology should:

- provide consistency, transparency and auditability,
- make use of all evidence about a defence's performance in whatever format it appears, be it precise measurements, expert judgements, statistical data or model predictions,
- explicitly consider the uncertainty associated with measurements and expert judgements (Hall *et al.*, 1998),
- provide an indication of the dependability of the condition assessment by reviewing the quality of the available information and the process by which it was generated,
- combine as much site-specific data as exists with generic knowledge about the performance of different types of defence,
- optimise resource use by comparing the cost of obtaining evidence with the reduction in uncertainty it provides, whilst also considering the consequences of failure of the defence,
- allow easy incorporation into a risk-based management framework to allow risk-based prioritisation of works and monitoring, and,
- allow integration into an asset management database and show how a defence performs through time, allowing the engineer to identify damage patterns and likely failure modes.

A new probabilistic approach to condition characterisation of flood and coastal defences that fulfils the above objectives is described in Chapter 5.

2.9.3. Decision-support and asset management

A detailed study of the core guidance manuals (Section 2.5 and Appendix C) and discussion with experts and practitioners (Section 2.8) has shown that flood defence management in England and Wales involves numerous organisations interacting at a multitude of levels. The sheer scale of the operation, the many stakeholders and the vast quantities of monitoring data, places a heavy information processing burden on decision-makers which can result in less efficient decisions being undertaken. A number of decision-making processes that could benefit from improved decision-support have been identified:

- transparency and auditability of decision-making,
- evidence gathering,
- level of information analysis,
- handling uncertainty,
- systems and strategic management, and,
- funding methods.

These are discussed in the following sections.

Transparency and auditability in decision-making

The introduction of DEFRA's project appraisal guidance has improved the transparency in decision-making for large scale capital projects that are applying for a DEFRA grant. SMPs and CFMPs also provide improved transparency and auditability in the decision-making process on a catchment of coastal cell basis. The guidance provides a degree of technical freedom allowing decision-makers to use the most up to date techniques, whilst providing a more standardised decision-making process. This improved transparency results from the need to externalise the decision-making process to other stakeholders (in many cases this includes the general public). Minor capital projects or those that do not qualify for grant aid are not required to follow the procedures laid down by DEFRA. However, capital projects are usually appraised using this guidance. The FDMM, despite having an appraisal methodology, is generally not used for capital project appraisal.

Guidance for the appraisal of maintenance projects is given by the FDMM. The extent to which these are implemented varies across England and Wales. Whilst the FDMM does standardise the decision-making process for maintenance, its deficiencies have made it unpopular amongst practitioners. Maintenance and inspection decisions are made through a process of negotiation between Flood Defence Officers and managers. Whilst the FDOs have a good understanding of the behaviour of their part of the flood defence system their decision-making processes need to be recorded and captured in a transparent and auditable manner. This reduces the reliance on the FDOs as 'corporate memory' which is increasingly important as organisational downsizing and the outsourcing of work becomes more commonplace. Because the decision-making process has not been properly formalised, it is often not clear how the decision to invest is made, what evidence is used to support these decisions and why the decision was made to gather this evidence. Despite gathering a great deal of evidence on the performance of flood defences, the connection between this evidence and identifying the need for an intervention is frequently not made.

Decision-making needs to be more integrated. Whilst the decision-making process for larger projects has become more transparent, these decisions are still considered and appraised independently to maintenance decisions. A more holistic approach that identifies and allows comparison of the benefits to the system as a whole from a number of different types of decision should be considered.

Evidence of performance

The process of options analysis and evaluation in flood defence management involves assembling and manipulating vast quantities of evidence. This evidence will appear in a range of formats, including dense numerical model results, textual evidence in technical reports, analogous cases, expert judgements, and perceptions and value judgements from the wider stakeholder group. In other words the evidence appears at very different levels of granularity and does not lend itself to being compressed into a single format. Whilst there may be a large volume of information relating to a decision, it is on the whole only of partial relevance, incomplete and sometimes conflicting. Decision-makers are therefore facing intense information processing demands (Hall and Davis, 2001) and this requires the structuring of evidence so it can be used efficiently. A number of key issues on the evidence of the performance of flood defence systems have been identified, these relate to issues of data collection, usage and analysis, the use of expert judgement and the importance of flood risk as a measure of flood defence system performance.

Evidence on system performance needs to be presented in a clear and structured manner to enable all stakeholders to communicate better with each other. If these approaches are not consistent and to some extent formalised, there is potential for monitoring and remediation resources to be mis-directed.

Information collection and analysis

Evidence on system performance needs to be of a density, frequency and quality appropriate to the decision being supported. The level of analysis should also compliment the scale and importance of the decision being supported. Currently detailed information is collected and then discretised to support decisions when a more approximate data collection strategy would suffice. Conversely, expert judgements can be used to generate precise failure probabilities that are then used to justify enormous investments. This approach compliments that of a tiered risk assessment framework described in Section 2.9.1.

Expert judgement

Because of the scarcity and cost of obtaining monitoring information, there has historically been a major input of expert judgement in condition assessment. Decision-making in the FDMM and FCDPAG series relies heavily on expert judgement. A degree of expert judgement is inevitable and the experience of experts, such as the FDOs, can provide a lot of useful information on the performance of a system. However, it is important that these judgements are explained and justified. This ensures that there is transparency in the decision-making process and that the decision-maker is aware of where these judgements came from, their limitations and the uncertainty associated with them. Integration of both qualitative and quantitative evidence provides a broader picture of system performance.

Flood risk

Section 2.9.1 identified the need for improved flood risk assessment because of its importance as a measure of the performance of a flood defence system. Measures such as flood risk should be considered in the context of other parts of the flood defence system to provide a richer overview of system performance.

Uncertainty

Whenever making decisions, it is important to consider uncertainty. Uncertainties in flood defence management arise from a number of areas, including natural randomness, such as wave loadings (inherent uncertainty) or lack of knowledge, for example in geotechnical conditions (epistemic uncertainty). Presently, uncertainty is primarily handled through the use of engineering judgement and factors of safety. Although the FDMM has no methodology to incorporate uncertainty in the decision-making process, more explicit approaches to uncertainty are being promoted (DEFRA, 2000b). Being able to identify and quantify uncertainty enables the decision-maker to target resources more efficiently and optimise data gathering and monitoring strategies. Uncertainties and how they influenced the decision-making process need to be recorded in an auditable and transparent manner.

Systems and strategic management

In order to put into place effective maintenance and investment strategies it is important to be able to understand how these strategies will alter the performance of not just a particular asset but the entire system. The FDMM provides no guidelines or framework for considering systems effects. The importance of this is recognised in SMPs (DEFRA, 2001c), FCDPAG2 (DEFRA, 2001b) and CFMPs which promote the implementation of strategic approaches, however, little guidance is provided as to how this should be done.

The interconnectivity of the elements of a flood defence system is not considered sufficiently at any level of decision-making. System connectivity needs to be identified at the all levels of the system. This needs to start at the lowest levels, where the condition of a flood defence is based on an average assessment of all the individual structural elements. The interconnectivity between these elements and neighbouring defences needs to be considered. This connectivity is influenced by shared properties, such as geotechnical variables, shared loadings and support offered by neighbouring defences and elements. The relationship between this physical infrastructure and other parts of the system such as flood warning systems, emergency response and education programmes also needs to be considered.

Identifying the connectivity of the system provides a better understanding of system behaviour and the relative importance of different parts of the system and can help to improve resource allocation. There are only a limited number of generic investment decisions for flood defence infrastructure, these are:

- do nothing,
- structural repairs (periodic maintenance)

- cleaning, mowing *etc.* (routine maintenance)
- capital works,
- increase or decrease inspection frequency, and,
- increase or decrease data collection.

To be able to identify the correct choice of intervention, these decisions need to be compared within the same methodology. For example, would an increase in maintenance be a more efficient long term use of resources than a capital project to replace an asset? Or, would increasing the density of data collection increase certainty in the performance of an asset to a level whereby a more informed decision can be made? At a higher level in flood defence management, this investment in the physical needs to be weighed against the importance of operating a reliable flood warning system and public education programmes.

Understanding the connectivity between system components allows a more holistic approach to decision-making to be achieved because the system wide impacts of decisions can be monitored. For example, what change in the overall system performance is expected when more money is spent on flood warning? Will the reduction in cost resulting from mowing embankments twice a year instead of four times a year decrease the overall system performance as a result of less efficient conveyance? Being able to answer these questions is of great importance when justifying and prioritising investment decisions as investment can be targeted where it is most needed in the system.

Funding

Funding allocation is reviewed on an annual basis making long term planning difficult. The present system of providing block grants to individual projects can lead to a less efficient use of funding. Whilst impetus for change with regards to financing flood defence projects needs to come from policy makers in national and local government, a decision-support methodology needs to be able to demonstrate the benefits that could be achieved from a change in funding strategies. Economics need to be incorporated into any decision-support tool so that the flow of funds and any change in performance resulting from investment decisions can be demonstrated.

Key needs in decision-support

In the light of the issues discussed above, the following key needs for decision support have been identified:

- to provide a model that considers the processes enacted by the sub-systems and their interconnectivity,
- to assemble evidence about asset condition and performance from diverse sources and represent it in a common and coherent model,
- to provide a model that integrates risk assessments with other measures of system performance,

- to externalize (to outside organisations and other decision stakeholders) expert judgements and decisions in a transparent and auditable manner,
- to provide a commentary on sources and implications of uncertainty in the evidence,
- to provide a platform for testing the implications of alternative asset management options (including data collection options), and,
- to facilitate dialogue between experts who specialise in different aspects of the flood defence systems and other decision stakeholders.

Whilst a quantitative flood risk assessment provides a useful and logical decision-making tool, it is inevitably incomplete and will exclude evidence of interest to the decision-maker because it appears in an inappropriate format or at an unsuitable scale. A performance-based approach to asset management that satisfies these criteria is described in Chapter 6.

2.10. SUMMARY

Since the Easter floods of 1998, flood defence management in England and Wales has been in a state of rapid evolution. This chapter has shown through analysis of the current guidance and structure of flood defence management that, coupled with the fact flood defence managers are under increasing pressure to use resources more efficiently, there is a cogent motive to improve decision-support techniques. Three key areas have been identified for improvement and these are:

- a quantitative risk assessment methodology that can be tailored to suit the decision being supported,
- a probabilistic condition characterisation methodology that explicitly considers failure modes and lends itself to integration within the aforementioned quantitative risk assessment methodology, and,
- a performance-based decision-support methodology that enables the risk assessment and condition characterisation to be integrated within the broader context of system performance.

These methodologies are described in Chapters 4, 5 and 6 respectively.

Chapter 3

Theoretical background to flood risk assessment

3.1. Overview

This Chapter explores the theoretical background to flood risk analysis. Two of the three areas identified for further research in Chapter 2 are improved methods for systems-based risk assessment and condition characterisation. A probabilistic method of assessing defence condition is a necessary step on the route to quantitative risk-based management of flood defence systems. The theory behind these two areas is explained in this Chapter, whilst much of the theory relating to the proposed decision-support methodology is introduced in Chapter 6.

The structure of this chapter is based on the source-pathway-receptor model (DETR *et al.*, 2000) which can be used to establish relationships between the sources of hazards (such as rain or waves), the pathway by which it is transmitted (such as over the floodplain) and their consequences (such as flooding of property). Originally developed to manage environmental hazards such as pollution, this framework is useful as it reflects the physical processes by which flooding occurs and deals explicitly with the impacts which concern the decision-maker (Pollard and Guy, 2001). With respect to flooding, it is clear that the risk is predominantly governed by the receptor. The source (precipitation, waves and tidal surges) cannot be controlled, the pathway (flood defences, and floodplain), if managed appropriately can be used to mitigate risk, but it is the receptor (people and property) that have the potential to be controlled the most (ICE, 2001). Figure 3.1 shows an example of a source-pathway-receptor relationship for a flood defence system. Rain, melting snow or coastal storminess leads to increased loading, this in turn results in direct overtopping of the defences, or possibly their failure. Failure of the defences results in flooding of residential and non-residential property damaging property and the economy. People are also affected with effects ranging from distress and inconvenience to illness or loss of life. The impacts on the natural environment from flooding can be positive or negative. This may involve the creation of important new wetlands, or the destruction of sensitive habits. These impacts may be long or short term.

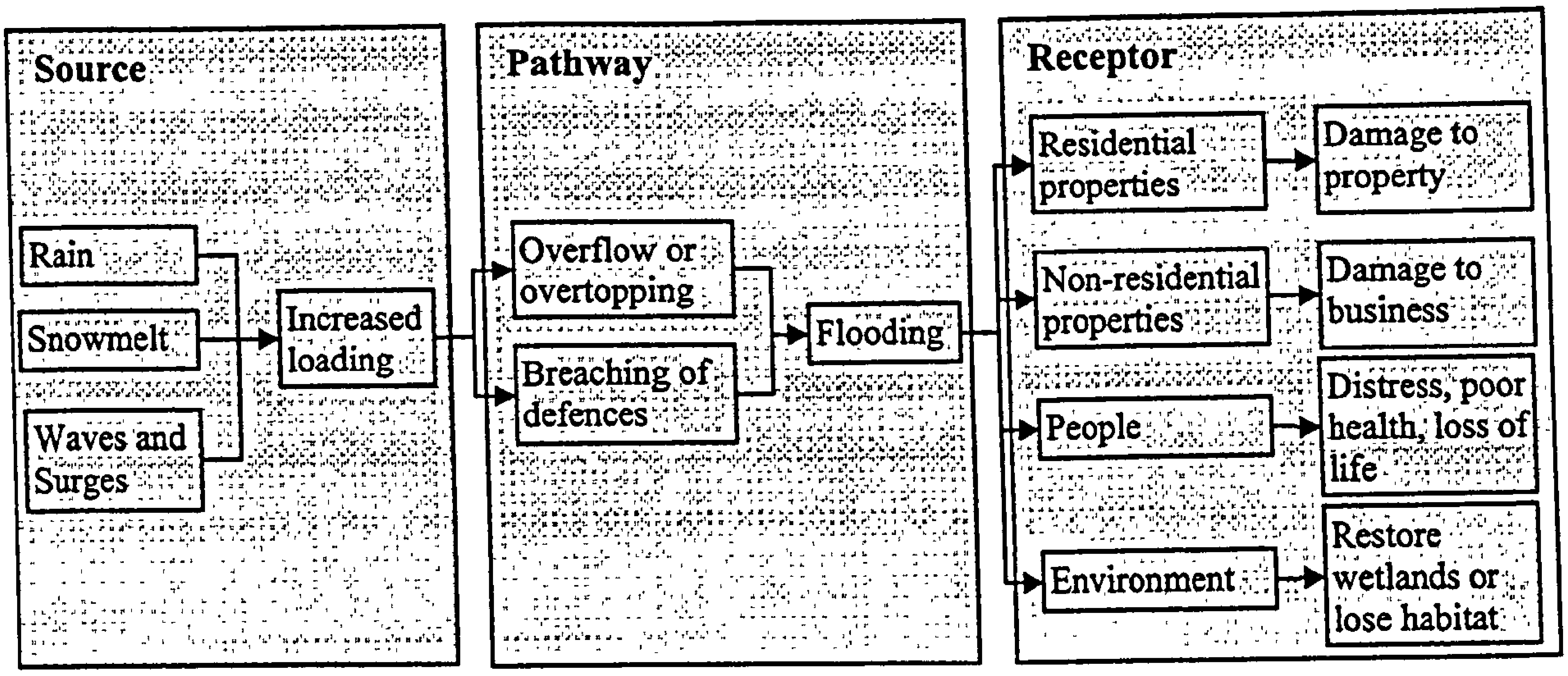

Figure 3.1 An example of a source-pathway-receptor relationship

This chapter describes the probabilistic risk assessment process from a theoretical point of view. Risk assessment and decision-making in flood defence management are characterised by uncertainty. Different types of uncertainty and how the techniques available to handle them are therefore reviewed in Section 3.2 prior to consideration of specific steps of a risk assessment. Methods used to model the source and estimate loads are described in Section 3.3. A large part of the research described in this thesis is on flood defence systems reliability. The pathway aspect of the risk assessment is described in Section 3.4. This section describes techniques to estimate defence failure probabilities. Section 3.5 discusses the impacts of flooding and how these can be measured and incorporated into a quantitative flood risk assessment.

3.2. UNCERTAINTY

Uncertainty is defined by the National Research Council (2002) as:

> *"...a general concept that reflects our lack of knowledge or sureness about something or someone, ranging from just short of complete sureness to an almost complete lack of conviction about an outcome."*

Early attempts to deal with uncertainty in flood risk management have been based on tradition. For example, the USACE (1996) would traditionally add approximately 3ft of freeboard as a factor of safety onto their embankments. In the UK (ICE, 2001) and the Netherlands (Pilarczyk, 1998) the previously highest recorded water level was used as the design event.

How a decision-maker manages uncertainty will often vary upon the situation. If the uncertainty is small, and the consequences of any variability are small the decision-maker may choose to ignore it in the analysis (Benjamin and Cornell, 1970). Where uncertainty is significant the decision-maker may account for it conservatively by applying a safety factor. Flood risk management requires consideration of uncertain processes and therefore requires appropriate techniques for handling uncertainty to ensure efficient resource use and transparent decision-making.

As identified in Chapter 2, decision-makers need tools that can use information in whatever format it appears and also identify and manage the uncertainty involved in their decision. It is useful to distinguish between different types of uncertainty. Hacking (1975) identifies two categories of uncertainty; inherent and epistemic uncertainty. Inherent uncertainties represent natural variability and randomness in samples and cannot be reduced (Paté-Cornell, 1996). Epistemic uncertainties are caused by lack of knowledge of the system and our ability to measure and model it and can therefore be changed as knowledge increases (Parry, 1996). Van Gelder (2000) proposes a classification of uncertainty appropriate for flood risk managers which will form the basis for the review in this Chapter (Figure 3.2).

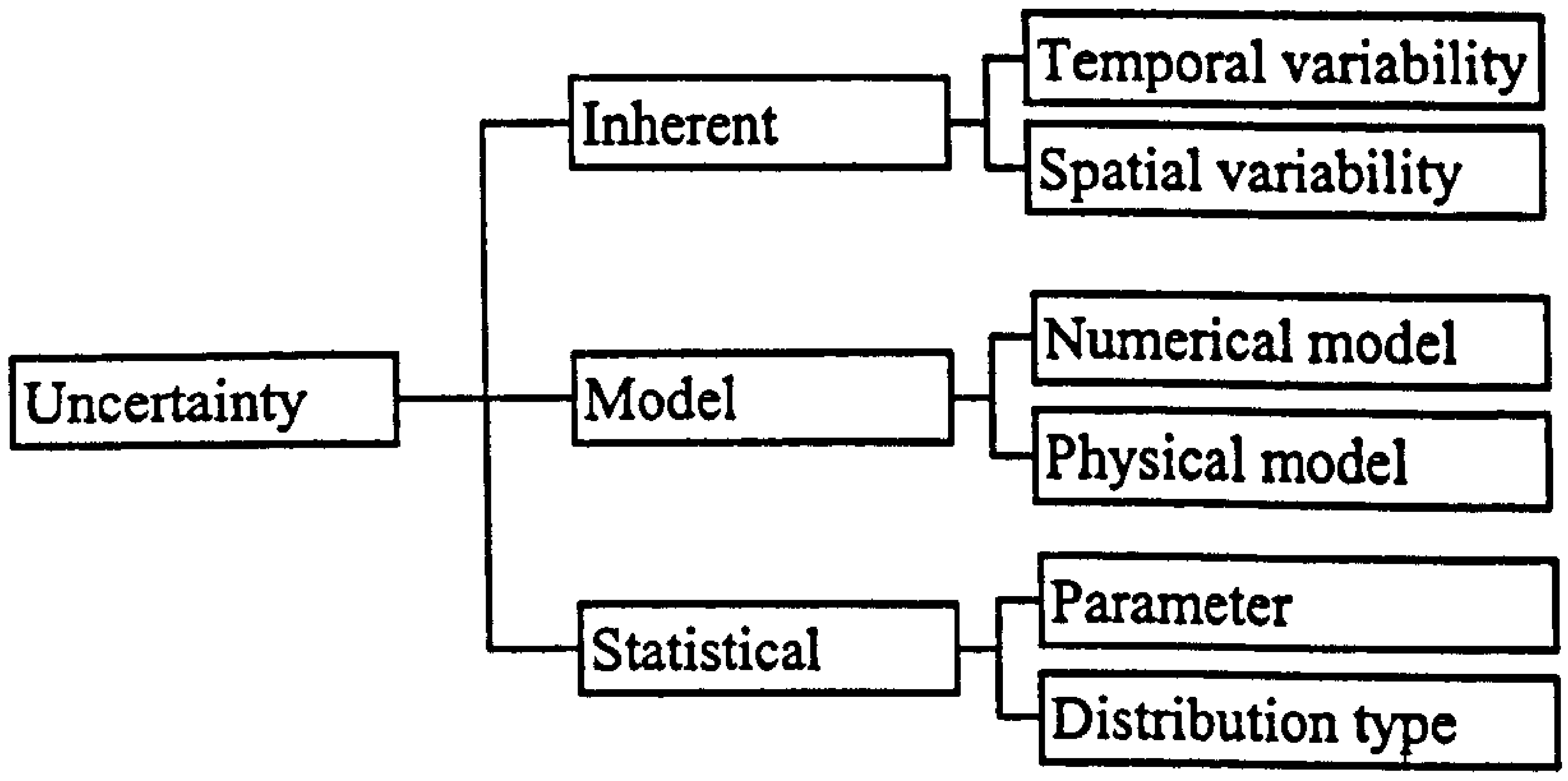

Figure 3.2 Classification of uncertainty in flood defence modelling (adapted from Van Gelder, 2000)

3.2.1. Inherent uncertainties

Inherent uncertainty, often known as random or natural variability, can be further categorised into temporal and spatial variability. Temporal inherent uncertainty represents the uncertainty associated with a process that varies with time, for example due to the uncertainty in predicting future wave heights. Spatial inherent uncertainty represents variations in the space of a process, for example due to variations in material strength along the length of a defence. It is useful to discern between these two types of inherent uncertainties (Vrijling and Van Gelder, 1998).

A realisation of a stochastic load that varies with time (for example individual wave heights) remains unpredictable as inherent uncertainty in time can not be reduced despite gathering unlimited data (Hora, 1996). However, acquisition of more data, for example on wave height, does mean that the certainty with which the predicted probability of realising a given wave height can be decreased (see Section 3.2.3 for more detail on statistical uncertainties).

Spatial variability can be described as stochastic processes in space. Conditions between the sampling points can be described by a probability distribution and autocorrelation function (Vrijling and Van Gelder, 2000). Spatial uncertainties can often be reduced by denser sampling strategies (Van Gelder, 2000). For example, there is only one realisation of material strength at any point along the length of a flood defence. Sampling this strength every 50m as opposed to 500m will act to decrease the uncertainty associated with the distribution and autocorrelation function used to describe the spatial variability (see Section 3.2.3 for more detail on statistical uncertainties).

3.2.2. Model uncertainty

Uncertainty in modelling can be considered as epistemic uncertainty (Zio and Apostolakis, 1996). Representing the real system as X, and the system model as X^*, the model uncertainty, N, is defined by Ang (1973) as:

$$X = NX^* \tag{3.1}$$

If the state of the real system is random, then the model, X^*, and uncertainty, N, will be random variables. Model uncertainty reflects the uncertainty associated with using a process model based on incomplete process knowledge, or data, to represent a system. Numerical models of physical processes are incomplete and physical models are subject to scaling effects (HR Wallingford, 2002).

3.2.3. Statistical uncertainty

Statistical uncertainty is a type of epistemic uncertainty. Statistical uncertainty can be further divided into parameter uncertainty and distribution uncertainty (Van Gelder, 2002). Parameter uncertainty (also known as statistical inference uncertainty) occurs when the parameters of a distribution are determined from a limited number of data. The amount of uncertainty is related to the size and variability of the dataset (Van Gelder, 2002).

Distribution uncertainty (also known as statistical model uncertainty) refers to the uncertainty that results from the selection of a particular statistical model to extrapolate a particular set of data (HR Wallingford, 2002). Judgement is used to select appropriate models and fitting techniques.

In terms of uncertainties in flood defence, it is clear that inherent uncertainties dominate the loading on flood defences, whilst epistemic uncertainties dominate structural response and how it is modelled.

3.2.4. Expressing and handling uncertainty

Uncertainty can be expressed in a number of ways (HR Wallingford, 1997, 2002, Klir and Folger, 1988 and Hall, 2003).

- Precise value: "crest height is 11.2m AOD (above ordnance datum)".
- Upper and lower bound: "the crest height is 11.2m ±0.3m AOD".
- Linguistic statement of deliberate vagueness such as "the defence is unlikely to fail".
- Unquantified ranking: "defence A is less likely to fail than defence B".
- Probabilistic value: "there is a 1% chance of overtopping".

These types of uncertainty can be captured using either probability or possibility distributions. These are discussed in the following sections.

Probabilistic handling of uncertainty

Many phenomena of concern to the flood risk manager are random. A probability distribution describes the variability of these phenomena. A probability distribution can be either discrete or continuous (Ditlevsen, 1981, Thoft-Christensen and Baker, 1982, Winkler, 1996 and Melchers, 1999). Historically, probability theory has been the primary tool for representing uncertainty in mathematical models (Dodagoudar and Venkatachalam, 2000). Knowledge of the probability of given flood loads allows a flood risk manager to balance the cost of investment in flood prevention against the expected impacts of flooding. This provides a more efficient solution than designing for the worst case loading which may be prohibitively expensive.

Probability theory

Probability theory is based on certain fundamental axioms, the most widely accepted being those of Kolmogorov:

- every event A in a sample space has probability $0 \leq P(A) \leq 1$,
- the probability of the inevitable event, S, is $P(S)=1$, consequently the probability of the impossible event, R, is $P(R)=0$, and
- for two events that are mutually exclusive (*i.e.* realisation of A_1 precludes the realisation of A_2):

$$P(A_1 \cup A_2) = P(A_1) + P(A_2) \tag{3.2}$$

For n events $A_1,\ldots A_n$ that are not mutually exclusive:

$$P=\left(\bigcup_{i=1}^{n} A_i\right)=\sum_{i=1}^{n} P(A_i)-\sum_{i<j} P(A_i \cap A_j)+\sum_{i<j<k} P(A_i \cap A_j \cap A_k)-\ldots+(-1)^{n+1} P(A_1 \ldots \cap A_n) \tag{3.3}$$

If there is a degree of dependence between two events, A and B, then conditional probabilities are used. The conditional probability of A occurring assuming B has occurred is given by:

$$P(A \mid B)=\frac{P(A \cap B)}{P(B)} \tag{3.4}$$

It therefore follows that if A and B are independent events, then:

$$P(A \mid B) = P(A) \text{ and } P(A \cap B) = P(A).P(B) \tag{3.5}$$

For mutually exclusive and collectively exhaustive events,

$$P(B) = \sum_{i=1}^{n} P(B \mid A_i).P(A_i) \tag{3.6}$$

which is known as the theorem of total probability. For a more detailed overview of probability the reader is referred to Ang and Tang (1975) and Grimmett and Stirzaker (2001).

Statistical inference

There are a number of methods that can be used to estimate the parameters of probability distribution functions. Whilst a comprehensive review is beyond the scope of this thesis an introduction to classical statistical methods and Bayesian inference are provided below. Detailed descriptions of these methods and a number of others are available from many sources, including (but not exclusively) Ang and Tang, 1975, Van Gelder, 2000, Bedford and Cooke, 2001.

A distribution of probability can be described by its moments, the first two being the mean or expectation, $E(X)$, and the variance, $var(X)$ (Ditlevsen, 1981), given by:

$$E(X) \equiv \mu_x = \int_{-\infty}^{\infty} x f_x(x)dx \tag{3.7}$$

$$\mathrm{var}(X) \equiv E(X-\mu_x)^2 \equiv \sigma_x^2 = \int_{-\infty}^{\infty} (x-\mu_x)^2 f_x(x)dx \tag{3.8}$$

The third order moment, or skewness represents the degree and direction of symmetry of the probability distribution and is defined as:

$$E(X-\mu_x)^3 = \int_{-\infty}^{\infty} (x-\mu_x)^3 f_x(x)dx \tag{3.9}$$

The use of probabilistic distributions to describe loading conditions is well established (CUR and TAW, 1990, CIRIA and CUR, 1991, HR Wallingford and Lancaster University, 2000). The approach generally applied in the UK to estimate coastal loads is that of Hawkes *et al.* (2002) which fits existing records to generate joint exceedance probabilities of wave height and water level.

Method of moments

The method of moments sets the moments of a distribution function equal to those of the observed sample. The sample moments are calculated using:

$$E(X) \equiv \mu_x = \frac{1}{n}\sum_{i=1}^{n} x_i \tag{3.10}$$

$$M_i(X) \equiv E(X-\mu_x)^i \equiv \sigma_x^2 = \frac{1}{n}\sum_{j=1}^{n} (x_j-\mu)^i \tag{3.11}$$

where M_i represents the ith moment and n the number of samples.

Method of maximum likelihood

An improvement on the method of moments is the method of maximum likelihood as it provides unbiased (*i.e.* its expected value is equal to the true value) parameter estimates. The likelihood can be regarded as representing the information about θ coming from the observed data. The maximum likelihood is therefore the value that makes the observed data most likely. For the random variable with density $f(x|\theta)$ where θ denotes the parameters describing the distribution of a random variable X with x_n observations. θ is chosen to maximise the likelihood function, L, of observing the data set $x_1,\ldots x_n$:

$$L(\theta|x_1,\ldots,x_n)=\prod_{i=1}^{n} f(x_i|\theta) \tag{3.12}$$

The method of maximum likelihood is most suitable to large sample sizes as the shape of the function is less biased by rarely observed events (Van Gelder, 2000).

Bayes' theorem

Bayes' theorem (Bayes, 1763 and see also Box and Tiao, 1973 and O'Hagan, 1994) provides a technique for updating a distribution (known as a prior distribution) in the light of new information to generate a so called posterior distribution. For an event, B, and a collection of events $A_1,\ldots,A_n$ contained in the set Ω such that $A_i \cap A_j = \emptyset$ whenever $i \neq j$ (mutually exclusive events), and $A_1 \cup \ldots \cup A_n \subset \Omega$ (collectively exhaustive events):

$$P(A_i|B)=\frac{P(B|A_i).P(A_i)}{\sum_{j=1}^{n} P(B|A_j).P(A_j)} \tag{3.13}$$

where $P(A_i)$ is called the prior probability, $P(A_i|B)$ is the posterior probability and $P(B|A_i)$ is the likelihood. This is also valid for continuous variables:

$$f(\theta|x)=\frac{f(x|\theta).f(\theta)}{\int_{-\infty}^{\infty} f(x|\theta).f(\theta)\,d\theta} \tag{3.14}$$

where X and θ are defined as for the method of maximum likelihood with joint probability density function $f(x,\theta)$ and corresponding conditional densities $f(x|\theta)$ and $f(\theta|x)$ and $f(\theta)=\int f(x,\theta)dx$ the marginal density of θ.

The prior distribution is supposed to represent knowledge about parameters before the outcome of, for example, an experiment is known. Ideally the prior probability distribution is elicited on the basis of available information, judgement or past experience. The empirical Bayesian method is to use the available data (perhaps pooled from many experiments) to select an appropriate prior distribution and then a classical estimation procedure (such as the method of moments or maximum likelihood) to estimate the distribution parameters (Bedford and Cooke, 2001). Where the form of this distribution is unknown, a non-informative distribution can be used. A number of methods of obtaining non-informative priors are now introduced.

A non-informative prior distribution that can be thought of as completely neutral is uniform over all the parameter space (a completely flat prior is likely to be improper as its integral will be infinite, therefore a diffuse prior such as $\mu \sim N(0,\sigma)$ where $\sigma \rightarrow \infty$ can be used). Note that the term non-informative is misleading as even in this case, the prior is stating that all the values are equally likely. Flat priors do not always give a proper posterior density after updating and there are consistency problems with their use (Efron, 1978).

Jeffreys (1961) proposed that a non-informative prior, $p(\theta)$, be invariant under a one-to-one parameter transformation. The non-informative prior of $p(\theta)$ can be shown to be $p(\theta)=I(\theta)^{½}$ (Box and Tiao, 1973) where $I(\cdot)$ is Fisher's (1925) measure of information about θ in a random variable, x:

$$I(\theta) = -\underset{x|\theta}{E}\left(\frac{d^2 \log f(x|\theta)}{d\theta^2}\right) \tag{3.15}$$

where $\underset{x|\theta}{E}(\cdot)$ is the expectation of $(\cdot)$ with respect to the distribution of the likelihood function $f(x|\theta)$. This measure of information quantifies how much is learned about θ from the observation x by calculating the expected curvature of the likelihood function (*i.e.* a measure of the sensitivity of the likelihood to θ). However, a prior distribution should only represent prior information and a weakness of Jeffreys' prior is that it is dependent on the form of the data (O'Hagan, 1994).

An alternative approach to constructing minimally informative priors is to maximise entropy. The entropy, $H(f)$ of the density $f(\theta)$ is defined as (Jaynes, 1957, 1963):

$$H(f) = -\int_{-\infty}^{\infty} f(\theta)\log f(\theta)d\theta \tag{3.16}$$

and can be thought of as a measure of how uninformative $f(\theta)$ is about θ. To minimise information, a distribution of $f(\theta)$ can be found that maximises the entropy. Unlike Jeffreys' method, the maximum entropy method is not invariant to parameter transformations (O'Hagan, 1994). Without applying constraints the entropy method renders a uniform prior distribution. Kapur and Kesavan (1992) and Singh (1997) amongst others provide a detailed overview of the maximum entropy approach and provide a general solution for identifying a non-uniform prior.

Bayes' theorem can also be used to account for statistical model uncertainty by calculating the posterior probabilities for all competing models and providing a Bayesian discrimination procedure between competing models (Pericchi and Rodriguez-Iturbe, 1983).

Generalised Likelihood Uncertainty Estimation for measuring model uncertainty

The generalised likelihood uncertainty estimation (GLUE) technique was developed by Beven and Binley (1992) and Beven (2000) to estimate model and parameter uncertainty of models. It is essentially a Bayesian model weighting method (Howson and Urbach, 1993).

The GLUE technique recognises the fact that there is no single correct model/parameter set combination that describes a system. Often many different combinations of input parameters result in equivalent or near-equivalent system behaviour when performing. This is the concept of *equifinality*. The rational behind the GLUE technique is to assume all combinations of model structures and parameter sets could be a possible simulator of the system. The likelihood of each parameter set is assigned a value based on comparing the observed and predicted system response. Models that do not represent the observed behaviour of the system are rejected and given a likelihood of zero. Therefore only behavioural simulations (simulations that exhibit behaviour similar to the system being modelled) are used to estimate the model uncertainty. The likelihood measures are used to weight the predictions of the remaining models and estimate uncertainty for the simulation. Likelihood values from different data can be combined as more data is collected. Dependency between parameters is accounted for as it is reflected in the likelihood value. The main processes of the GLUE procedure (Beven, 2000) are:

(1) choosing model(s) to be assessed in the analysis,
(2) selecting feasible input distributions for the input parameters,
(3) sampling the parameter space using Monte Carlo simulations to obtain random parameter sets,
(4) choosing an appropriate likelihood measure and setting a threshold for unacceptable (non-behavioural) parameter sets, and,
(5) identifying acceptable and non-acceptable simulations and weight each simulation by the likelihood value of the parameter set used and derive uncertainty bounds.

An initial sample space needs to be defined for the parameter values. The parameter space needs to be large enough to include simulations with a high likelihood, but not so large that meaningless model runs are simulated. This is frequently a subjective reflection of prior knowledge of the parameter values, however it is safest to start with a wide range as the Bayesian likelihood procedure refines the parameter range as more data is added (Beven and Binley, 1992). The sampling strategy should be chosen such that the parameter sets most likely to give a good representation of the system are chosen. In most GLUE applications, a uniform independent sampling of parameters in the parameter space is used because of its ease of use, however this is inefficient if large areas of the parameter space result in non representative simulations (Beven, 2001). The efficiency can be improved if a limited number of exploratory simulations are used to identify local optima (simulations with high likelihoods). Further simulations are selected so that sampling is denser in these regions (Werner and Khu, 2002).

The likelihood measure, L_e, should equate to zero for all outputs that do no reflect the behaviour of the system, and its value should increase as the similarity to the system being modelled increases.

A number of approaches are suggested by Beven and Binley (1992), one such approach calculates the model efficiency using Equation 3.17.

$$L_e = \left(1 - \sigma_e^2 / \sigma_o^2\right), \ \sigma_e^2 < \sigma_o^2 \qquad (3.17)$$

where σ_o^2 measures the variance of the observations and σ_e^2 measures the variance of the residuals defined as:

$$\sigma_e^2 = \frac{1}{n}\sum_{t=1}^{n}(Q_t - \hat{Q}_t\{\underline{\Theta}, \underline{Y}\})^2 \qquad (3.18)$$

where n is the number of time steps, Q_t is the observed value at time t and $\hat{Q}_t\{\underline{\Theta}, \underline{Y}\}$ is the simulated value given parameters $\underline{\Theta}$ and input data $\underline{Y}$. If the simulated values form a perfect fit with the observed data, the likelihood is unity. For a fit no better than assuming the mean of the data is known (*i.e.* $\sigma_e^2 = \sigma_o^2$) it takes a value of zero (Nash and Sutcliffe, 1970). It is important to note that good calibration data is needed in order to assess how well the model performs in comparison to observed system behaviour.

The uncertainty is calculated by describing the likelihood values as a probabilistic weighting function. A distribution of the model predictions may then be generated, thereby allowing variance, percentiles and other measures of uncertainty to be calculated.

As more data becomes available, the likelihood weights may be updated, Beven and Binley (1992) suggest the use of Bayes' theorem.

$$L_p(\underline{\Theta} \mid \underline{y}) = L_y(\underline{\Theta} \mid \underline{y}) L_o(\underline{\Theta}) \qquad (3.19)$$

where $L_o(\underline{\Theta})$ is the prior likelihood distribution, $L_y(\underline{\Theta}|\underline{y})$ is the calculated likelihood given the new observations y, and $L_p(\underline{\Theta}_y)$ is the posterior likelihood distribution of the parameter set. The GLUE therefore depends on model test data which may not be available (Hall and Anderson, 2002).

Possibilistic handling of uncertainty

Whilst exceedance probabilities are naturally suited to describe hydraulic loads, flood defence management is not concerned only with numerical information. As described in Chapter 2, expert opinion plays an important role in decision-making in flood defence. In information-scarce situations, such as predicting the effects of climate change, the *possibility* of an event occurring is considered rather than attempting to assign a probability, requiring some form of expert judgement to elicit a description of a variable. This description can be captured in the form of a fuzzy set.

A fuzzy set is defined by a membership function μ_A: $X \rightarrow [0,1]$ where μ_A is the degree of membership of any element of X in A with a value of 1 representing full membership. Fuzzy sets were introduced by Zadeh (1978 and 1983) and are used to describe the possibility of membership. A possibility distribution represents the inherent vagueness in linguistic terms and was introduced to describe sets whose membership criteria are imprecise (Klir and Folger, 1988) thereby generalising crisp set theory in which the membership of μ_A would be limited to a value of either 0 or 1.

An example of a group of membership functions is shown in Figure 3.3. This membership function has been constructed to represent the possibility of a man being described as 'short', 'average', 'tall' or 'very tall'. People are often referred to as 'tall', but this linguistic term is inherently imprecise. This imprecision can be captured in a fuzzy set that shows the possibility, μ, of someone being of a certain height. For example, to be called 'tall', the lowest possible height is 1.8m (approximately 5ft 10") and the highest is 2.04m (6ft 8") with the most possible height for a man to be described as 'tall' ranges from 6ft 2" to 6ft 5". It is impossible for a man to be described as 'tall' outside the upper and lower ranges, however, it should be noted that at a height of 2m (6ft 7") it is still possible to describe a man as 'tall', however there is a greater possibility of him being described as 'very tall'.

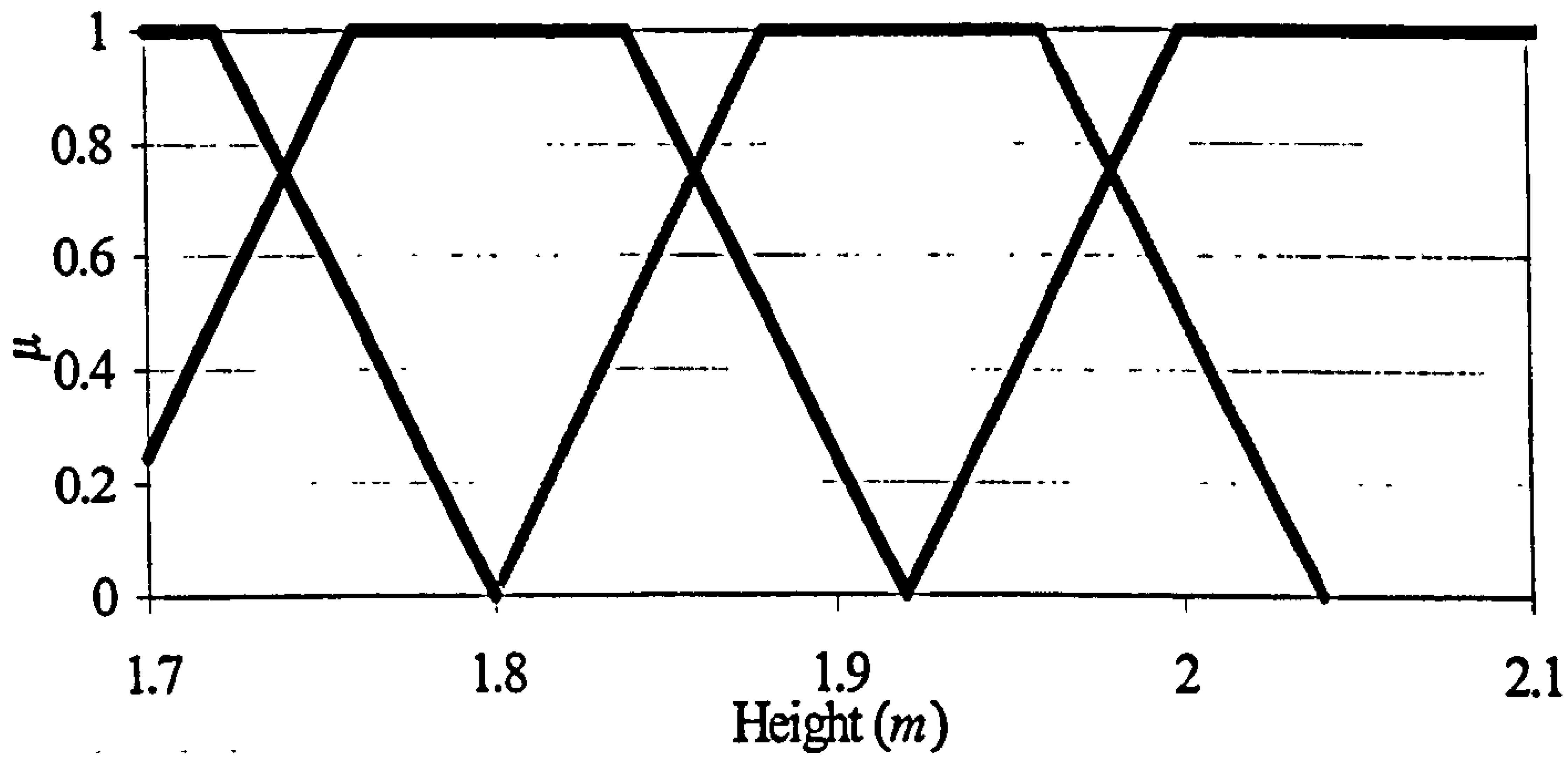

Figure 3.3 Example of a trapezoidal membership function describing a man's height (from left to right) as either 'short', 'average', 'tall' or 'very tall'

Possibility (or fuzzy set) theory

Whereas probability theory deals with the randomness of future events, possibility theory deals with the possibility of events (Casciati and Faravelli, 1991). Fuzzy sets can be operated in the same manner as crisp sets such that:

$$\mu_{\bar{A}}(x) = 1 - \mu_A(x) \tag{3.20}$$

$$\mu_{A \cup B}(x) = \max[\mu_A(x), \mu_B(x)] \tag{3.21}$$

$$\mu_{A \cap B}(x) = \min[\mu_A(x), \mu_B(x)] \tag{3.22}$$

In order for any function of this form to qualify as either a fuzzy union or intersection, it must satisfy the following conditions (Klir and Folger, 1988):

- $f(0,0)=0$; $f(0,1)=f(1,0)=f(1,1)=1$; *i.e.* f behaves as it would for a crisp set,

- $f(a,b)=f(b,a)$; *i.e.* f is commutative,
- $a \leq a'$ and $b \leq b'$, then $f(a,b) \leq f(a',b')$; *i.e.* f is monotonic, and,
- $f(f(a,b), c) = f(a, f(b,c))$; *i.e.* f is associative.

where f represents the function of intersection or union of two sets. It is often desirable in certain applications to consider the following additional requirements:

- f is a continuous function (a small increase in the membership grade in either set A or B doe not therefore produce a large change in the membership of $A \cup B$), and,
- $f(a,a) = a$; *i.e.* f is idempotent (the union of any set with itself produces the same set).

Evidence theory

Shafer's (1976) mathematical theory of evidence, which is an extension of Dempster's original theory (1968), is a tool that is used to represent ignorance, or imprecision in evidence, through belief and plausibility measures. A belief measure is a function:

$$Bel\colon \wp(X) \rightarrow [0,1] \tag{3.23}$$

where $\wp(X)$ is the power set consisting of all the subsets of X. The belief function assigns to each crisp subset of X a number in the interval [0,1] and satisfies the axioms (Klir and Yuan, 1995): .

- $Bel(\emptyset) = 0$ and $Bel(X) = 1$;
- for every $A, B \in \wp(X)$ if $A \subseteq B$ then $Bel(A) \leq Bel(B)$;
- if the sequence $(A_i \in \wp(X) \mid i \in \mathrm{N})$ of the subset of X is monotonic (*i.e.* $A_1 \subseteq A_2 \subseteq .. A_n$ or $A_1 \supseteq A_2 \supseteq .. A_n$) then $\lim_{i\to\infty} Bel(\mathrm{A_i}) = Bel(\lim_{i\to\infty} \mathrm{A_i})$, and,
- for every $n \in \mathrm{N}$ and every collection of subsets of X:

$$Bel(A_1 \cup A_2 \cup ... A_n) \geq \sum_i Bel(A_i) - \sum_{i<j} Bel(A_i \cap A_j) + ... + (-1)^{n+1} Bel(A_1 \cap A_2 \cap ... A_n) \tag{3.24}$$

Associated with each belief measure is a plausibility measure, Pl, defined as:

$$Pl(A) = 1 - Bel(\overline{A}) \tag{3.25}$$

However, plausibility measures can be defined independently as:

$$Pl\colon \wp(X) \rightarrow [0,1] \tag{3.26}$$

and satisfying the first three axioms that satisfy the belief measure and for every $n \in \mathrm{N}$ and every collection of subsets of X:

$$Pl(A_1 \cap A_2 \cap ... A_n) \geq \sum_i Pl(A_i) - \sum_{i<j} Pl(A_i \cup A_j) + ... + (-1)^{n+1} Pl(A_1 \cup A_2 \cup ... A_n) \tag{3.27}$$

Every belief measure and its plausibility measure can be expressed in terms of the basic probability assignment which is a function:

$$m\colon \wp(X) \rightarrow [0,1] \tag{3.28}$$

such that $m(\emptyset) = 0$ and $\sum_{A\in\wp(X)} m(A) = 1$ (3.29) where $m(A)$ is the degree of evidence, or probability mass, supporting the claim that a specific element of X belongs to the set A, but not to any subset of A. A belief and plausibility function can therefore be defined by:

$$Bel(A) = \sum_{B \subseteq A} m(B) \text{ for all } A \in \wp(X) \tag{3.30}$$

$$Pl(A) = \sum_{B \cap A \neq \varnothing} m(B) \text{ for all } A \in \wp(X) \tag{3.31}$$

$Bel(A)$ is a lower bound on a family of probability measures and $Pl(A)$ is an upper bound.

Dempster's rule of combination can be used to combine two sources of evidence, B and C, where the probability mass of the combined evidence, $m(A)$, is given by:

$$m(A) = \frac{\sum_{B \cap C = A} m_1(B) m_2(C)}{\sum_{B \cap C \neq A} m_1(B) m_2(C)} \tag{3.32}$$

This rule of combination assumes that the two sources of evidence are 'distinct', in that the knowledge of one piece of evidence does not induce non-vacuous belief in the truth of the other (Smets, 1990).

Dubois and Prade (1991) extended evidence theory to provide a mechanism for projecting uncertain information through a function of the form $\underline{y}=f(\underline{x})$. The uncertain dependency between $(x_1, x_2 \ldots x_n)$ can be expressed in terms of a random relation which is a random set $(\Re, \rho)$ on the Cartesian product $X_1 \times X_2 \ldots X_n$ such that:

$$m(A) = \sum_{A = y(R_i)} \rho(R_i)\ ,\ \Im = \{y(R_i) \mid R_i \in \Re\}, y(R_i) = \{y(x) \mid x \in R_i\} \qquad \text{for all } A \in \Im \tag{3.33}$$

where $(\Im, m)$ defines the range of y.

Hall (2003) demonstrates an application of this extension to evidence theory using Owen's equation to estimate bounds on wave overtopping rates (HR Wallingford, 1980). The water level, wave height and wave period are described using joint measurements, the crest level is represented as an interval value. Two model coefficients are described using fuzzy sets (rather than taking pre-assigned deterministic values), chosen to represent the uncertainty associated with using a model based on experimental data as a site specific tool. Because of the complexity of visualising the random set that is generated, cumulative belief and plausibility functions are plotted. These estimate bounds of probability, providing a representation of the uncertainty in the estimate of overtopping volumes.

Second order random variables to describe uncertainty

Burmaster and Wilson (1996) proposed the use of second-order random variables to separate variability from uncertainty. Instead of describing uncertainty using a single random variable, individual parameters that describe random variability are mapped through a probability distribution representing the uncertainty in the description of this parameter. This may be related to the equipment used to measure the evidence, or the expert judgement involved in estimating the variability. A parameter is now described by moments that are themselves expressed in terms of a distribution, for example μ=N(5,0.5) and σ=N(1,0.5). This allows variability and other uncertainties, which have different management implications, to be separated.

3.3. SOURCE: FLOOD LOAD ESTIMATION

The source is "synonymous with hazard and refers to a situation with potential for harm (for example, heavy rainfall, strong winds, storm surge *etc.*" (ICE, 2001). This section provides an introduction to techniques used to estimate the magnitude and frequency of storm events. The accuracy and type of output of the load estimation has an impact on the uncertainty associated with a flood risk assessment. Knowledge of loadings is required to enable a flood risk manager to estimate the loadings on the system and therefore the defence failure probabilities.

3.3.1. Fluvial flooding

The majority of river floods in the UK are due to intense precipitation. Catchments in colder or mountainous regions will experience flooding caused by snowmelt. On rare occasions, landslides or dam failure can also lead to flooding. Catchment properties, such as its size, shape and degree of urbanisation and forestation and the part of the catchment subjected to the intense rainfall influence the size of the flood flow in the river.

Flood estimation using data

The Flood Estimation Handbook (CEHW, 1999) is the standard for hydrological modelling in England and Wales. Other approaches to rainfall-runoff modelling have been developed and some of these are discussed briefly here. The Flood Estimation Handbook identifies two methods used to estimate peak flood flows and their associated probabilities:

(1) Statistical analysis of existing flow records, and,

(2) Rainfall-runoff methods.

The statistical method requires a long dataset. Annual maximum flows are ranked and the probability of occurrence of the flow, X, is given by:

$$P(X \geq Q) = \left(\frac{r-a}{N+1-2a}\right) \tag{3.34}$$

where Q is the flow rate for a given event, r is the ranking of the flood event (a rank of 1 represents the highest flow in the record), N is the number of annual maxima and a is a constant for particular probability density functions. The Weibull distribution that is used in the United States to calculate flood frequencies assigns a=0 (Helsel and Hirsch, 2002). However, a more unbiased value of a (Chadwick and Morfett, 1993) commonly used to plot an extreme value distribution is given by Gringorten (1963) as 0.44. The return period, T_R, of the flood is:

$$T_R = \frac{1}{P(X \geq Q)} \tag{3.35}$$

It is recommended that the method is not used for extrapolating flows exceeding the 1:200 year event (CEHW, 1999).

There are many rainfall-runoff methods (Beven, 2000), the most common of these in England and Wales is based around the unit hydrograph (Sherman, 1932) and is used when little or no flow data is available. Catchment characteristics are extracted from the Flood Estimation Handbook (CEHW, 1999) to calculate the time to peak of the unit hydrograph which relates the effective rainfall and the storm runoff. Using more standard characteristics obtained using methods in the Flood Estimation Handbook, the flow rate can be calculated for a given return period.

Flood estimation using process-based models

Process based models aim to simulate the processes involved in hydrology by using suitable equations. An overview of the processes involved in hillslope hydrology is shown in Figure 3.4 and Figure 3.5.

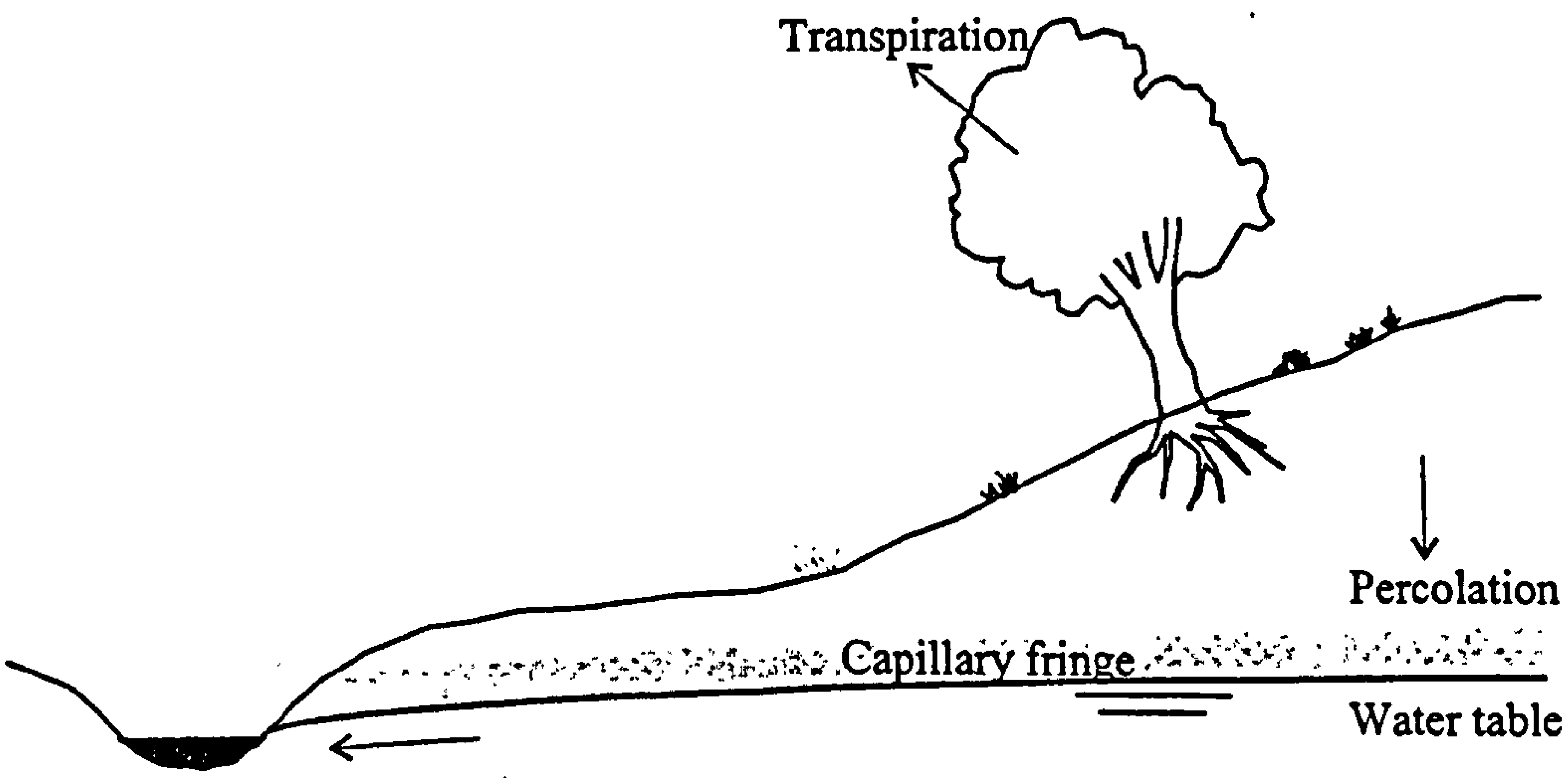

Figure 3.4 Processes in a perceptual model of hillslope hydrology between storms (Beven, 2000)

Freeze and Harlan (1969) laid down a blueprint for process-based models by linking the equations for surface and sub-surface flow. This is still followed by the majority of models despite the necessary simplification of the processes shown in Figure 3.5 (Beven, 2000). Processes such as preferential flow cannot be adequately described despite attempts to do so (Bronstert and Plate, 1997).

The Systeme Hydrologique European (SHE) (Abbott *et al.*, 1986) is a grid based model. These elements are linked by surface runoff and groundwater flow components. Each grid element has a specified parameter set describing subsurface flow, vegetation, overland roughness, channel flow and snowmelt.

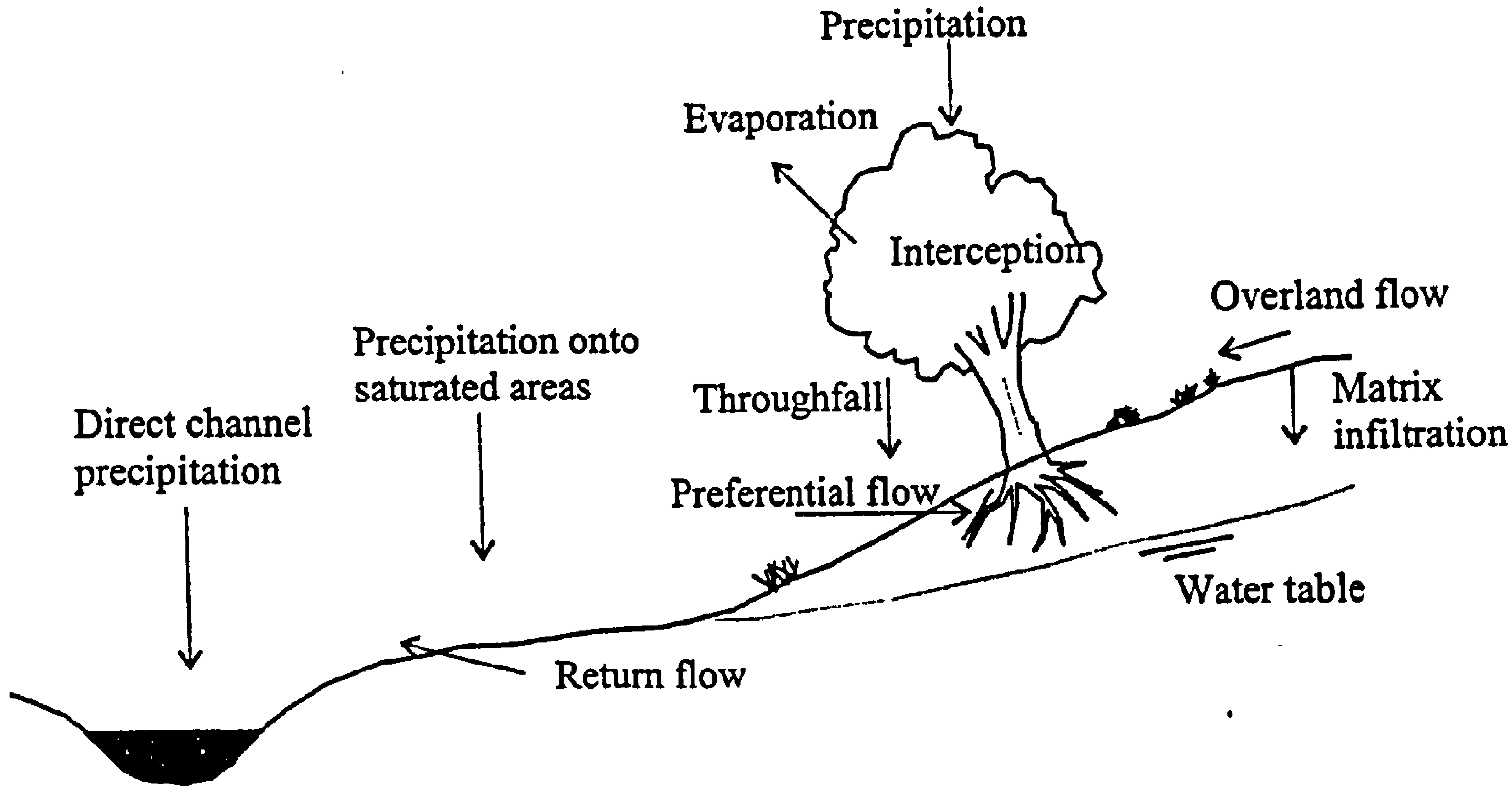

Figure 3.5 Processes in a perceptual model of hillslope hydrology during storms (Beven, 2000)

The Institute of Hydrology Distributed Model (IHDM) (Calver and Wood, 1995) discretises the catchment based on hillslope planes based on flow lines such that any lateral exchange of water between elements can be neglected. If the flow follows the form of the surface then this discretisation may be performed by a topographical analysis, however a 2-D model such as SHE is required if the geology is such that this assumption no longer holds (Beven, 2000).

Flood estimation using similarity and distribution function models

TOPMODEL (Beven *et al.*, 1995) takes a different approach to process-based models and assumes that all points in the catchment with the same topography index respond in the same manner. This index represents the propensity of any point in the catchment to become saturated (Beven and Kirkby, 1979). The reliance of this model on fewer parameters gives it an advantage over more parameterised models (Fleming, 2002), however it should be noted that as with the IHDM it is not always appropriate to parameterise the flow regime using topography.

Hydrodynamic modelling

There are two aspects to hydrodynamic modelling, in-bank river channel modelling and floodplain modelling. It is noted that hydraulic modelling of flow through the floodplain should technically be included in the pathway domain, however, it is briefly introduced in this section because it is a natural continuation of hydrological modelling and is not a feature of the novel research in this thesis.

Hydraulic models have a varying degree of complexity. Cluckie and Owens (1987) note that advances in hydrometeorology, computing and remote sensing have allowed major advances to take place in the design and implementation of extreme flood prediction. Integrated rainfall-runoff and flood routing has been implemented to a limited extent in England and Wales (Cluckie and

Han, 2000), however this does not yet incorporate real-time inundation analysis. Some commercial software does contain integrated hydrological modelling routines, but it is usual for upstream hydrographs to be used as initial inputs to hydraulic models.

Flow is usually modelled using a volume storage concept. This is based on conserving the volume of water flowing between two sections of river. Khatibi and Haywood (2002) proposed a categorisation of approaches to fluvial modelling based on the level of information revealed by a particular approach. The defining characteristics of each model category are given in Table 3.1. A detailed review of modelling techniques is beyond the scope of this thesis. However, a number of the more frequently used modelling methods are briefly described below.

Table 3.1 Categorisation of fluvial modelling approaches based on the concept of volume storage (Khatibi and Haywood, 2002)

Hydrodynamic routing models	Distributed prism/wedge storage Conservation of mass/momentum Physically meaningful parameters Extensive data required
Kinematic routing models	Distributed prism/wedge storage Approximating mass/momentun Physically meaningful parameters Ample data required
Hydrological routing models	Distributed prism storage Distributed layout Conservation of mass Some parameters
Conceptual models	A conceptual control volume Anput/output boundaries Conservation of mass Extensive parameters
Blackbox models	A lumped control volume Input/output boundaries Mathematical formulations Not conserving mass/momentum
Empirical model	A selection of points Regression equations
Heuristic rules	A selection of points 'What if' conditions
Rules of thumb	A single point No mathematics

The essential problem in analysing flow is shown in Figure 3.6. The Navier-Stokes equations apply at a single point, *P*, in a fluid. At this point the governing equation for the streamwise motion of a small element is (Knight and Shiono, 1996):

$$\rho\left[V\frac{\delta U}{\delta y}+W\frac{\delta U}{\delta z}\right]=\rho g\sin\theta+\frac{\delta\tau_{yx}}{\delta y}+\frac{\delta\tau_{zx}}{\delta z}$$

$$\underset{\text{(vorticity)}}{\text{Secondary flows}} = \text{Weight force} + \underset{\text{(lateral)} + \text{(vertical)}}{\text{Reynolds stresses}} \tag{3.36}$$

where $\{U,V,W\}$ are velocity components in the $\{x,y,z\}$ directions, ρ is the fluid density, θ is the slope, g is the gravitational acceleration and τ_{yx} and τ_{zx} are the Reynolds stresses on planes perpendicular to the y and z directions respectively.

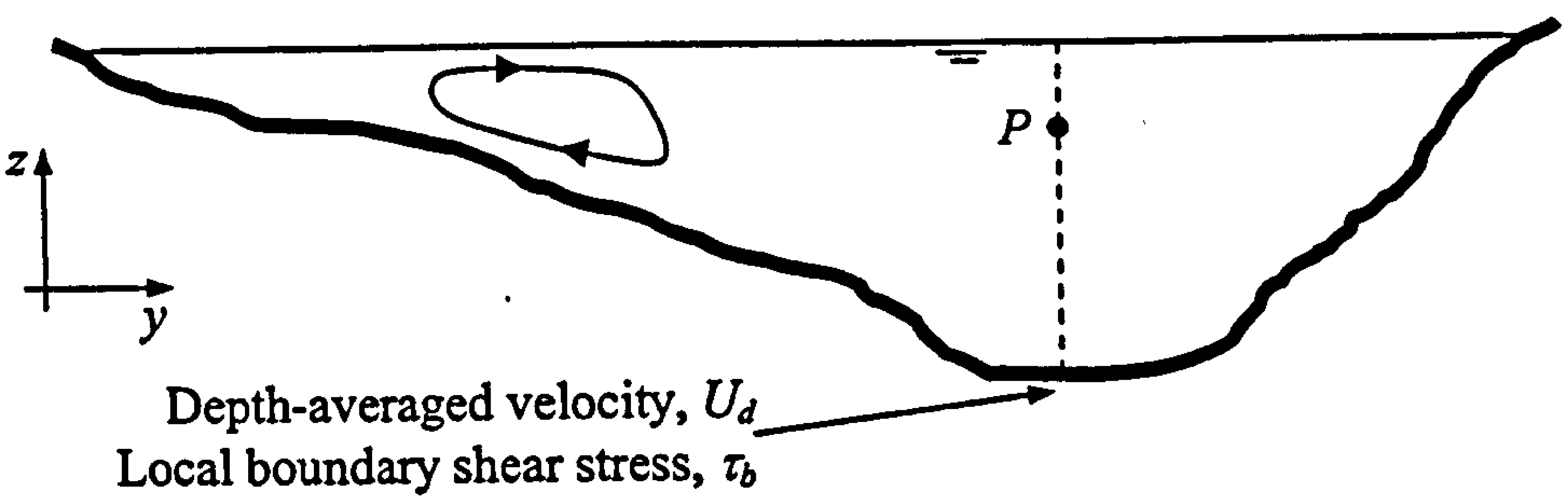

Figure 3.6 Flow in a natural channel

One dimensional models, so called because the one dimensional hydrodynamic St. Venant equations (Equation 3.37 and 3.38) for mass and momentum are solved along a series of cross-sections used to represent the river (Samuels, 1990, Ervine and MacLeod, 1999).

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t}=q \tag{3.37}$$

$$S_0-\frac{n^2P^{4/3}Q^2}{A^{10/3}}=0 \tag{3.38}$$

where Q is the volumetric flow rate in the channel, A the cross sectional area of the flow, q the flow into the channel from other sources (*i.e.* from the floodplain or possibly tributary channels), S_0 the down-slope of the bed, n the Manning's coefficient of friction, P the wetted perimeter of the flow, and h the flow depth.

Flow over embankments and through the floodplain is modelled using spill equations and structures can be accounted for using energy loss equations. Despite the simplifications of the physical process, they can provide reasonable estimates of water level (Fleming, 2002). Commercial packages include ISIS, MIKE 11, ONDA HYDRO 1-D and HEC-RAS.

Preistnall *et al.* (2000) suggested a simplified inundation modelling technique that involves intersecting a plane representing the flood water level with a Digital Elevation Model (DEM) to provide an estimate of flood extent.

LISFLOOD-FP (Bates and De Roo, 2000) provides an extension to one dimensional modelling of the river by modelling the floodplain as a 2D raster model. Flow in the channel is modelled using Equations 3.37 and 3.38, similarly floodplain flow is described in terms of momentum and continuity, discretised over a grid of square cells. This allows the model to represent 2D dynamic flow fields on the floodplain. Flow between cells is simply a function of the free surface height between the cells.

$$\frac{dh^{i,j}}{dt}=\frac{Q_x^{i-1,j}-Q_x^{i,j}+Q_y^{i,j-1}-Q_y^{i,j}}{\Delta x\Delta y} \tag{3.39}$$

$$Q_x^{i,j}=\frac{h_{flow}^{5/3}}{n}\left(\frac{h^{i-1,j}-h^{i,j}}{\Delta x}\right)^{1/2}\Delta y \tag{3.40}$$

where $h^{i,j}$ is the water free surface height at the node (i,j), Δx and Δy are the cell dimensions, n is the effective grid scale Manning's friction coefficient for the floodplain, and Q_x and Q_y describe the volumetric flow rates between floodplain cells. Q_y is defined analogously to Equation 3.38. The flow depth, h_{flow}, represents the depth through which water can flow between two cells, and is defined as the difference between the highest water free surface in the two cells and the highest bed elevation. This method provides performance comparable to that of more complex, two dimensional finite element models (Bates *et al.*, 2002).

Two dimensional models not only divide the channel cross-section into more areas, but also include processes that are excluded from one-dimensional flow equations. These include the lateral shear and secondary flows in addition to the usually dominant bed friction. The depth-averaged form of the Navier-Stokes equations is used to describe the direction and magnitude of flood water in the river and the floodplain (Shiono and Knight, 1991):

$$\rho gHS_0-\frac{1}{8}\rho fU_d^2\left(1+\frac{1}{s^2}\right)^{1/2}+\frac{\delta}{\delta y}\left\{\rho\lambda H^2\left(\frac{f}{8}\right)^{1/2}U_d\frac{\delta U_d}{\delta y}\right\}=\frac{\delta}{\delta y}[H(\rho UV)_d] \tag{3.41}$$

where s is the channel side slope and U_d is the depth mean velocity defined by:

$$U_d=\frac{1}{H}\int_0^H U\,dz \tag{3.42}$$

A popular commercial package in the UK is TELEMAC2D.

Three dimensional models take the analysis a stage further and involve solving a three dimensional Navier-Stokes equation (for example Younis, 1996, Falconer and Chen, 1996). Several commercial packages exist such as TELEMAC3D, DELFT3D and HYDRO 3-D which can model three dimensional flows caused by stratification, wind and waves.

3.3.2. Coastal flooding

The majority of coastal flooding is usually a result of low-lying land (for example, large areas of East Anglia in the UK) being inundated by storm surges (Smith and Ward, 1998). The likelihood of inundation can be increased at estuaries when a high river level encounters a storm surge in the sea, as occurred in the river Thames in 1928 (Brooks and Glasspoole, 1928). Inundation and damage may be caused directly by extreme wave conditions and in some parts of the world, tsunami are a real threat (Bascom, 1959).

Morphology

The intensity of coastal floods is influenced by the shape of the coastline and near shore bathymetry (Pugh, 1987). Areas such as the North Sea cause storm surges to be funnelled, resulting in a significant increase in water level (Ward, 1978). The characteristics of the seabed and coastline influence the degree of shoaling, refraction, diffraction and breaking of any wave attacking the shore and any coastal defence (McConnell, 1998). Bathymetry, through the alteration of wave loadings, therefore impacts on flood risk.

Modelling of coastal morphology plays an important role in coastal flood risk management (Reeve and McCue, 1997, Hall *et al.*, 2000). Coastal erosion results in reduced beach protection thereby increasing the flood risk. However, this erosion may also provide natural protection in the form of beach nourishment downdrift of the erosion site.

A detailed review of morphological modelling is outside the scope of this thesis. This section aims to provide an overview of a number of tools and methods available to flood risk managers. More detailed descriptions are available from many sources, including (but not exclusively) Komar (1998), Kamphuis (1999), Dean and Dalrymple (2001) and USACE (2002).

Methods of predicting shoreline movement are often made using a "one-line" model that predicts the position of a single contour line along a stretch of coast using a continuity equation for the sediment transport, S:

$$\frac{dS_x}{dx}+\frac{dS_y}{dy}+h\frac{dy}{dt}=0 \tag{3.43}$$

where S_x and S_y are the sediment transport components in the x and y direction in time t over a vertical range of sediment transport h (Figure 3.7).

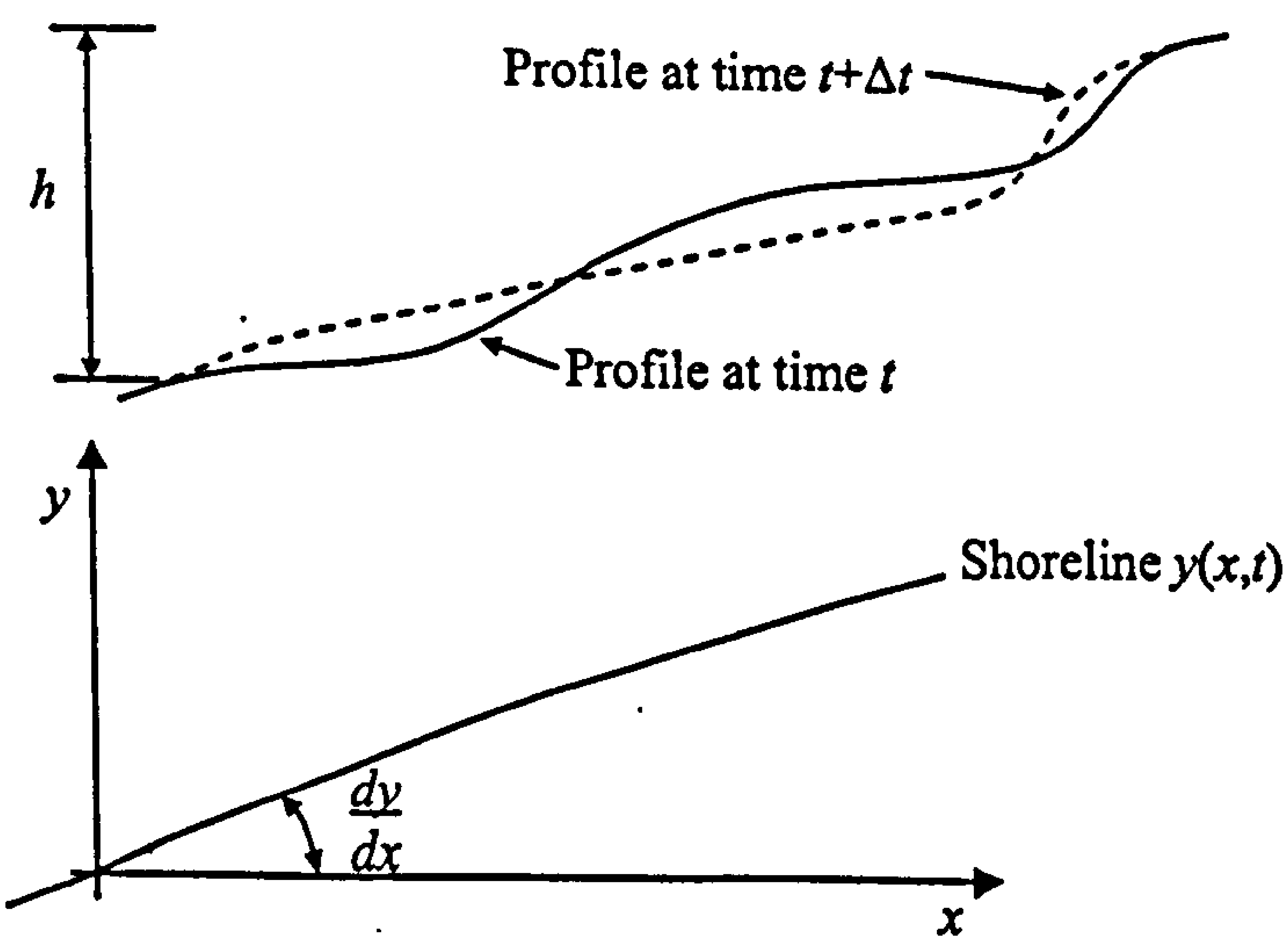

Figure 3.7 Definition of variables for the continuity equation for sediment transport (Reeve and McCue, 1997)

One line models have been extended to n-line models that describe horizontal evolution of the beach at a number of points across the profile (Bakker, 1968).

Stochastic methods (*eg.* Reeve, 1995, Johnson and Hall, 2002) perform a set of simulations to gauge the likely variation in the future shoreline rather than a purely deterministic prediction. Statistical-dynamical methods, proposed by Reeve and Fleming (1997), introduce an additional factor $F(x,t)$ that represents net long term contribution of processes other than mean long shore transport (*eg.* sediment sources, sinks). This is used to 'fit' the traditional one-line model to historical data. The one-line model combined with the forcing factor is subsequently used to predict bounds on the evolution of the coastline. Payo *et al.* (2002) have extended the one-line model to include time-varying boundary conditions.

De Vriend (1991) argues that long term prediction of evolution requires an alternative approach because long term trends in coastal evolution are "a weak residual of a very 'noisy' signal of short term variability". This has been addressed through behaviour-oriented (or phenomenological) modelling (De Vriend *et al.*, 1993). This maps the observed behaviour (both from historical records and other models) onto a simple mathematical model that exhibits the same behaviour within a given range of spatial and temporal scales.

The erosion of dunes and cliffs is important as these can be a large source of sediment. Dune erosion is usually modelled using Vellinga's (CUR and TAW, 1991) equations. An erosion profile is established based on storm conditions. The volume of dune eroded is assumed to correspond to the difference in volume between the initial beach profile and assumed storm profile (Figure 3.8).

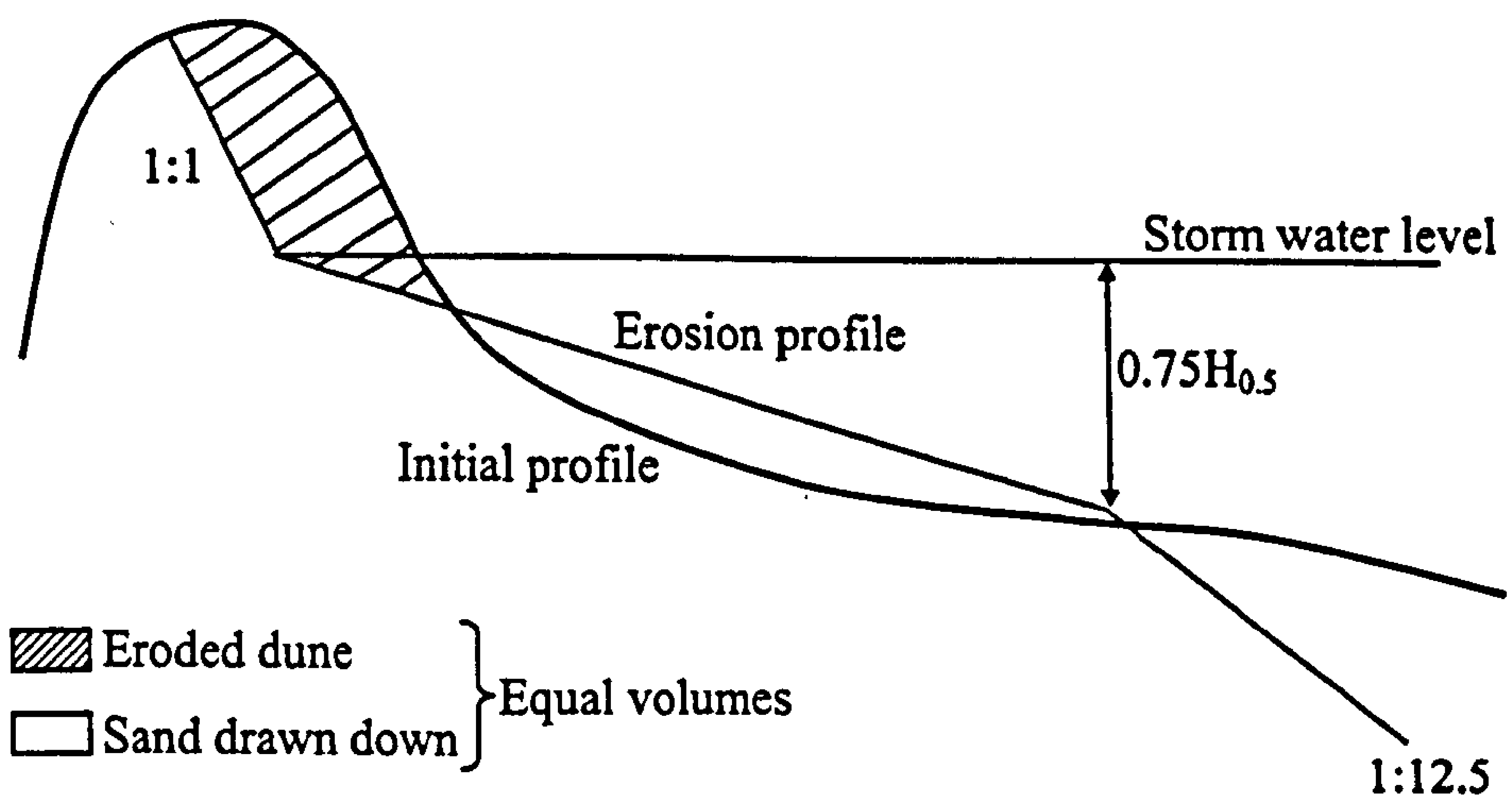

Figure 3.8 Dune system storm erosion/accretion balance (CIRIA, 1996)

The most common approach to assessing cliff erosion is based on extrapolating future recession scenarios using historical cliff position records (Dolan *et al.*, 1991, Cowell *et al.*, 1997). The closed profile approach applied to dune erosion can also be applied to predicting cliff erosion. The Bruun profile can be used to assess the change in cross-shore profile resulting from sea level rise and shoreline evolution (Dean, 1991). Recent advances have seen a move towards process-based modelling of cliff erosion (Meadowcroft *et al.*, 1999, Walkden and Hall, 2002). These models consider the interactions between the underlying processes of cliff erosion, such as the cross-shore distribution of erosion, longshore sediment transport, shore platform erosion and cliff-face stability. This approach is more appropriate to longer term modelling than the reductionist (and computationally expensive) approach to cross-shore modelling proposed by Nairn and Southgate (1993). A more detailed review of cliff erosion modelling is provided in Lee and Clark (2002).

Estimation of magnitude and frequency

Coastal loadings are usually described in terms of the joint probability of wave height and water level (from a tide or storm surge) as the two are statistically dependent. This is partly due to the fact that meteorological conditions tend to produce both extreme water levels and waves at the same time. Primarily, it is because extreme waves are usually depth-limited as they come close to the shore. The highest near shore waves occur when extreme offshore wave conditions coincide with extreme water levels.

The method proposed by Hawkes *et al.* (2002) involves first measuring data on wave height, wave period and water level or hindcasting this information from wind data. Statistical distributions are then fitted to these. The dependence between wave height, water level and wave steepness is also fitted. Monte Carlo simulations are used to simulate a large number of wave heights, wave periods and water levels using the fitted distributions. These simulations can be used to calculate extreme wave heights, periods and water levels. A more thorough description of joint-probability methods is given by HR Wallingford and Lancaster University (2000). Methods of constructing probability distributions are discussed in Section 3.2.4.

Inundation modelling

Coastal inundation can be caused by a number of mechanisms; overtopping, overflow and defence breaching. HR Wallingford (1999) has produced a manual that is used to calculate overtopping volumes based on wave heights and water levels for different structure types. Breaching mechanisms are discussed more in Section 3.4 and in Appendix D.

The consideration of tidal and wave effects can add a further complication to modelling coastal floods, however, there are packages that specifically account for these effects such as MIKE 21, and many of the aforementioned 2D and 3D packages, such as DELFT3D and the TELEMAC series can also model coastal floods.

3.4. PATHWAY – FLOOD DEFENCE RESPONSE

The pathway is defined as the "*connection between a particular hazard being realised and the receptor that may be harmed*" (ICE, 2001). One of the most widely used intervention strategies available to the flood risk manager is the construction and maintenance of flood defence structures. To enable an assessment of flood risk a manager must have an understanding of how these defences behave under a range of loads and the likelihood of their failure. This section discusses methods used to analyse flood defence failure in order to obtain flood defence system failure probabilities. Appropriate techniques are adapted and used in Chapter 4 and 5 as part of a new approach to systems-based risk assessment and condition characterisation.

3.4.1. Flood defences and their failure mechanisms

Flood and coastal defences can fail by many different mechanisms. Each of these mechanisms may have several initiating events. In theory these mechanisms can all be described using limit state functions. However many are poorly understood and there may be limited information for their application to a given site. Failure of a defence is defined as an event that results in water finding a way over, under or through it. This may be a result of breaching, overtopping, overflow, seepage or piping. The failure of a flood defence is one of the key events of the pathway stage of the source-pathway-receptor model.

A review of failure mechanisms for both flood and coastal defences is provided in Appendix D. This review describes the processes that can lead to failure of a defence, and gives details of limit state functions used to describe failure mathematically and the key parameters that influence failure. These functions can be used to estimate failure probabilities using reliability theory (Section 3.4.2).

3.4.2. Reliability analysis

It has long been recognised that absolute safety cannot be achieved and so a degree of poor or uncertain performance must be accepted (Freudentahl, 1947, Pugsley, 1951, Torroja, 1958). Reliability originally came about to enable the rational treatment of uncertainties in structural design and provides a decision-making tool for selecting an appropriate compromise between the requirements of safety and economy (Freudentahl, 1956, Cornell, 1967, Benjamin and Lind, 1969, Cornell, 1969). Reliability theory can be used to estimate flood defence failure probabilities. Whilst this section provides a background into reliability theory, research presented later in this thesis demonstrates how reliability theory can be used to assess the condition of structures with uncertain evidence and be adapted to make predictions of their future performance in a manner that compliments a risk-based framework. The term structural reliability is defined by Thoft-Christensen and Baker (1982):

> *"a structure's ability to fulfil its design purpose for some specified time, or in a mathematical sense, it is the probability the structure will not attain each specified limit state (ultimate or serviceability) over a period of loading."*

There are five main steps to a reliability analysis (Oumeraci *et al.*, 2001).

(1) Identification of the system, its components and failure modes.

(2) Definition of limit state equations for these failure modes.

(3) Identification of limit state variables.

(4) Calculation of reliability for each failure mode.

(5) Calculation of bounds for system failure.

These concepts and some of the methods used to estimate reliability will be introduced over the next few sections. A simple reliability problem is outlined and the methods available for solving it are described in more detail in the following sections.

The general reliability equation takes the form shown by Equation 3.44:

$$g(\underline{x}) = R - S \tag{3.44}$$

Where $\underline{x}$ is a vector of random variables describing the system; R is the strength of the structure and S is the load on the structure. Failure occurs when $g(\underline{x}) \leq 0$, the probability of failure is therefore:

$$Pf = P(g(\underline{x}) \leq 0) = P(R(\underline{x}) - S(\underline{x}) \leq 0) = \int_{g(\underline{x}) \leq 0} f_x(\underline{x})dx \tag{3.45}$$

Where f_x denotes the probability density function of the boundary conditions. This is demonstrated graphically in Figure 3.9.

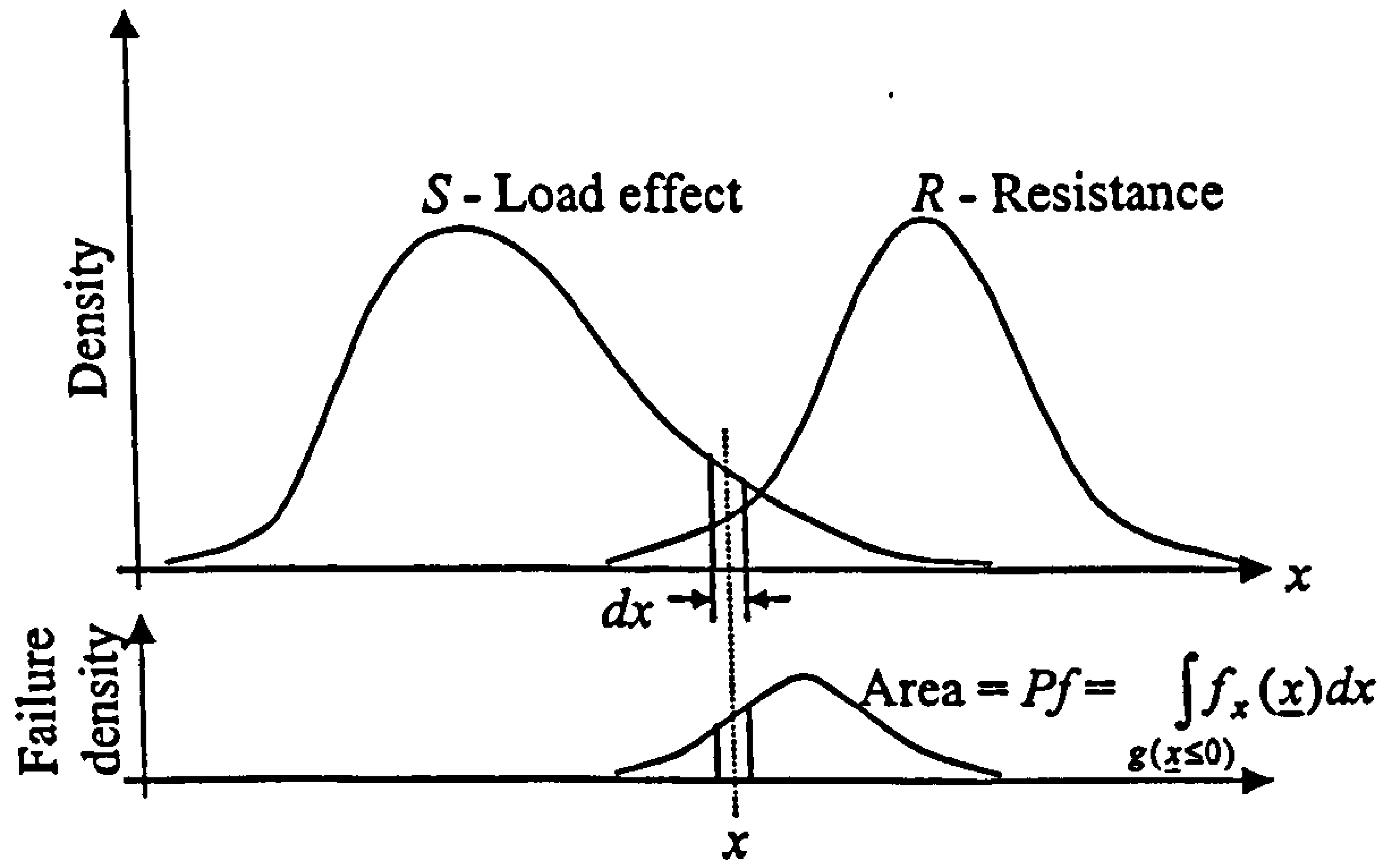

Figure 3.9 Basic reliability problem demonstrating load (S), resistance (R) and failure density

So-called level II and III reliability methods, summarised in the following sections, can be used to solve the integral in Equation 3.45 using numerical or analytical methods. A more detailed description of these methods is available from many sources, including (but not exclusively) Throft-Christensen and Baker (1982), Casciati and Faravelli (1991), Melchers (1995) and Ditlevsen and Madsen (1996).

Level II: First-Order Second Moment methods (FOSM)

A first-order method is so-called because it approximates the failure surface to a first order or linear function. First, considering the simplest case when both R and S are defined by two independent normal variables, the failure probability is described by:

$$Pf = P(R - S \le 0) = P(g(\underline{x}) \le 0) = \Phi\left(\frac{-(\mu_R - \mu_S)}{\sqrt{\sigma_R^2 + \sigma_S^2}}\right) = \Phi\left(\frac{\mu_g}{\sigma_g}\right) = \Phi(-\beta) \quad (3.46)$$

where μ and σ define the mean and variance of R and S; $\Phi(\cdot)$ is the standard normal distribution function and β is the *reliability* index (Cornell, 1969). As shown in Figure 3.10, β is a measure of the distance that μ_g is away from the boundary of the failure space in terms of standard deviation σ_g. If the failure function is non-linear then approximate values for μ_g and σ_g can be obtained by performing a Taylor series expansion:

$$g(\underline{x}) = g(x_1, x_2 ... x_n) \cong g(\mu_1, \mu_2 ... \mu_n) + \sum_{i=1}^{n} \frac{dg}{dx_i}(X_i - \mu_i) \quad (3.47)$$

where dg/dz_i is evaluated at (μ_1,μ_2... μ_n). However, this index is dependent on the linearisation expansion point (Ditlevsen, 1979). In order to minimise errors due to the linearisation, the expansion point should be a point on the failure surface rather than a mean point (Thoft-Christensen and Baker, 1982).

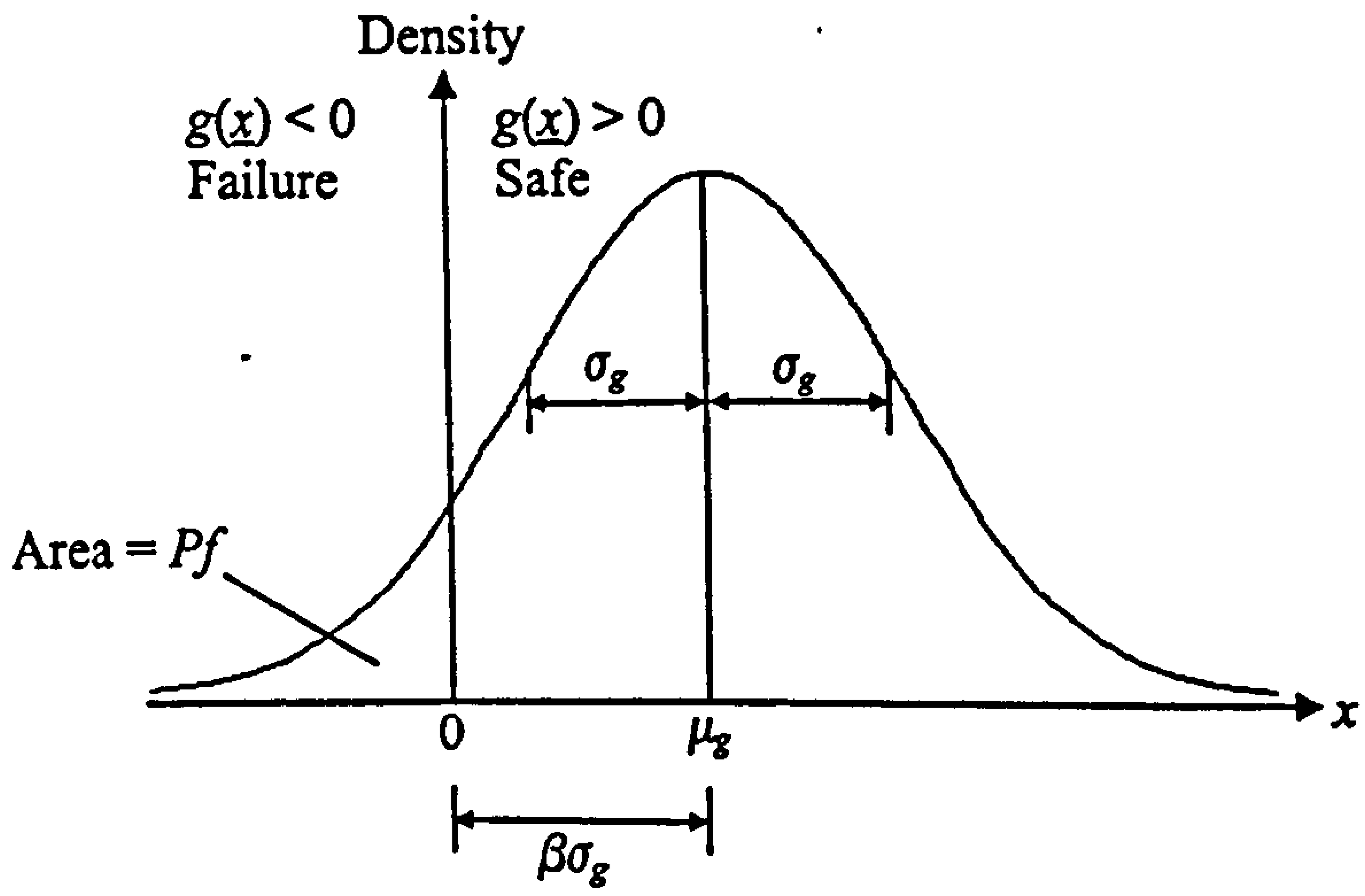

Figure 3.10 Definition of reliability index

This is readily extended to consist of more variables as long as the function $g(\underline{x})$ is linear and is defined as a normal distribution. Hasofer and Lind (1974) proposed a reliability index, β_{HL}, which is invariant to the choice of failure function. This is achieved by normalising the failure function variables onto a new co-ordinate system which has rotational symmetry with respect to the standard deviations. The reliability index, β_{HL}, is independent of the failure function because equivalent functions result in the same failure surface, $g(\underline{y})$=0. β_{HL} is defined as the shortest distance from the origin of the normalised co-ordinate system to the boundary of the failure surface. This is known as the design point (Figure 3.11).

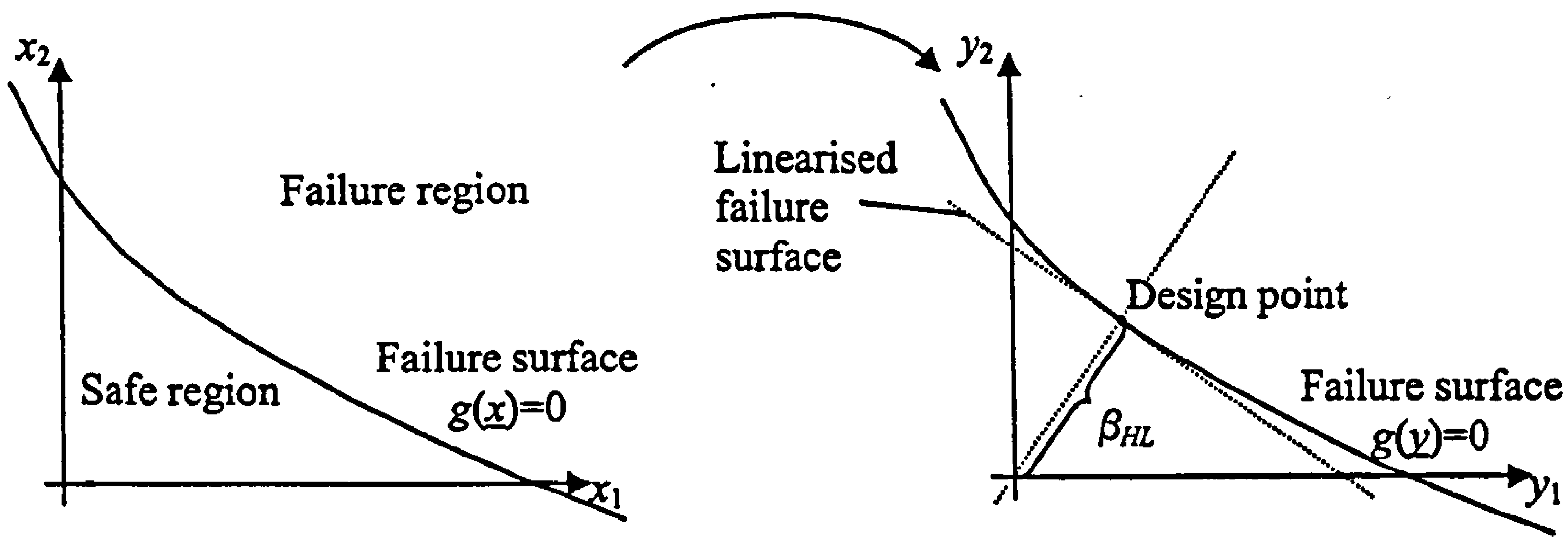

Figure 3.11 Mapping the failure variables onto a normalised co-ordinate system and defining β_{HL}, the Hasofer-Lind reliability index

Melchers (1999) formalises an algorithm to calculate β_{HL}.

(1) standardise the random variables $\underline{x}$ to the independent standardised normal variables $\underline{y}$ (Figure 3.11) using Equation 3.48,

$$y_i = \frac{x_i - \mu_{xi}}{\sigma_{xi}} \tag{3.48}$$

(2) transform $g(\underline{x})=0$ to $g(\underline{y})=0$,

(3) select initial trial point $\underline{y}^{(1)}$,

(4) compute $\beta^{(1)}$,

(5) let m=1,

(6) compute directional cosines α:

$$\alpha_i = \frac{-{dg_i}/{dy_i}}{K}, \quad K = \left[\sum_{i=1}^{n}\left({dg_i}/{dy_i}\right)^2\right]^{0.5} \tag{3.49}$$

(7) compute $g(\underline{y}^{(m)})$,

(8) compute $\underline{y}^{(m+1)}$:

$$\underline{y}^{(m+1)} = -\alpha^{(m)}\left(\beta^{(m)} + \frac{g\left(\underline{y}^{(m)}\right)}{l}\right) \tag{3.50}$$

(9) compute $\beta_{HL}{}^{(m)}$,

(10) check whether $y^{(m+1)}$ and/or $\beta_{HL}{}^{(m+1)}$ have stabilised; if not return to step 5 and increase m by unity,

(11) If optimising a design, calculate z_i the design points for the variables, or calculate *Pf*. β_{HL} is defined as:

$$\beta_{HL} = \min_{g(y)=0}\left(\sum_{i=1}^{n} y_i^2\right)^{0.5} \tag{3.51}$$

First order second moment methods can be extended to allow the use of random variables described by non-normal distributions (this is more commonly known as a First Order Reliability Method (FORM)).

To calculate the Hasofer-Lind reliability index, an additional transformation to convert these non-normal variables into normal variables is required. The Rosenblatt transformation (Rosenblatt, 1952 and Hochenbichler and Rackwitz, 1981) can transform a vector of variables into a set of uncorrelated, standardised and normally distributed variables. The transformation assumes that the original density and distribution functions correspond to the values of a normal distribution at the design point (see Figure 3.11). This is shown by Equation 3.52:

$$G(x_i^d) = \Phi\left(\frac{x_i^d - \mu'_{xi}}{\sigma'_{xi}}\right), \; g(x_i^d) = \frac{1}{\sigma'_{xi}}\left(\frac{x_i^d - \mu'_{xi}}{\sigma'_{xi}}\right) \tag{3.52}$$

where $\Phi(\cdot)$ is the standard normal cumulative distribution function, x^d_i represents the value of the basic variable x_i at the design point and μ'_{xi} and σ'_{xi} are the mean and standard deviations of the normal distribution. At each iteration, the mean and standard deviation need to be solved using Equation 3.53:

$$\sigma^d_{xi} = \frac{\phi(\Phi^{-1}(G(x_i))}{g(x_i^d)} \; , \; \mu'_{xi} = x_i^d - \Phi^{-1}(G(x_i^d))\sigma'_{xi} \tag{3.53}$$

where $\phi(\cdot)$ is the standard normal probability density function.

Level II: Second Order Reliability Method (SORM)

First order methods that approximate the failure surface (g=0) to a linear function become less accurate as the limit state function becomes more curved. Second order methods try to account for this by fitting a quadratic surface to the limit state function (Breitung, 1984 and Hohenbichler *et al.*, 1987). Second-order methods can be implemented using either sampling techniques or asymptotic approximation (Melchers, 1999).

The sampling method involves first estimating the probability using standard FORM analysis, the 'error' is then calculated by sampling the space between the first order and second order approximations of the limit state surface (Hohenbichler and Rackwitz, 1988).

The asymptotic approximation, for limit state functions that are not too non-linear and have only one design point, involves approximating the failure surface to an asymptotic function shown in Equation 3.54 (Breitung, 1984).

$$Pf \approx \Phi(-\beta)\sum_{j=1}^{k}\left[\prod_{i=1}^{n-1}(1-\beta\kappa_i)\right]^{-\frac{1}{2}} \; , \kappa_i = -\left[\delta^2 y_n \Big/ \delta y_i^2\right] \tag{3.54}$$

where κ_i is the ith principal curvature of the limit state at the design point.

Second order methods are clearly more complex than their first order counterparts and frequently require the use of sampling techniques to obtain a solution. Therefore, there is sometimes little advantage to be gained over using Level III methods.

Level III: Integration and simulation methods

Simulation techniques, often referred to as Monte Carlo simulations, involve sampling at random to artificially simulate a large number of experiments. This automatically deals with non-linearity in functions and non-normal random variables (Hammersley and Handcomb, 1966).

Direct sampling

This involves direct sampling from the basic variables and counting the number of failures. The failure probability, *Pf*, is therefore estimated using:

$$Pf \approx \frac{1}{N}\sum_{i=1}^{N} I[g(\underline{x}_i') \leq 0] \tag{3.55}$$

where N is the total number of simulations, $\underline{x}_i$ is the vector of the observation of the ith simulation and $I[g(\underline{x}'_i) \leq 0]$ is an indicator function that outputs 1 if $g(\underline{x}'_i) \leq 0$ and 0 if not. This method is robust as it can handle reliability calculations in which there is more than one design point. The number of simulations, N, required to obtain a given confidence level, C, in the failure probability, *Pf*, is (Broding *et al.*, 1964):

$$N > -\frac{\ln(1-C)}{Pf} \tag{3.56}$$

Therefore for a 99% confidence level when $Pf=10^{-3}$ over 4600 simulations are required. This calculation time for a Monte Carlo simulation is independent of the number of variables and can be improved by using importance sampling.

Importance sampling

The direct sampling approach described previously samples points uniformly, wasting considerable effort in sampling areas that are not in the region of interest, in a reliability analysis this will be the failure region. Techniques for overcoming this problem increase the density of sampling in the region of interest by ensuring a more efficient selection of random variables and hence increase the overall efficiency of the simulation. This requires selection of an 'importance-sampling' density function, $h_v(\underline{x})$ that approximates the failure function over the region of interest. The failure probability is now estimated using Equation 3.57:

$$Pf \approx \frac{1}{N}\sum_{i=1}^{N}\left(I\left[g(\underline{v}_i') \leq 0\right]\frac{f_x(\underline{v}_i')}{h_v(\underline{v}_i')}\right) \tag{3.57}$$

where $\underline{V}$ is a vector with a probability density function $h_v(\underline{v})$ and $\underline{v}'_i$ is a vector of sample values from the importance function $h_v(\cdot)$. Choosing appropriate importance functions can be challenging as it is important not to bias the estimate. Melchers (1989) suggests defining the importance function as $h_v = \Phi_n(\underline{v}, \underline{C}_v)$ where $\underline{C}_v$ is a diagonal matrix of σ_i^2 with the mean of $\underline{V}$ placed at $\underline{x}'$ (the maximum likelihood of $\underline{x}$). This can reduce the number of sample points required significantly

(Engelund and Rackwitz, 1993) and such an importance sampling function generates sample points that are unbiased with respect to each variable.

This and other methods of making Monte Carlo simulations more efficient, such as directional sampling, are described in more detail in Melchers (1999).

Random variables to estimate model uncertainty in a reliability analysis

There will always be a measure of epistemic uncertainty associated with any model of a system, however sophisticated it is. The two sources of uncertainty in limit state models are parameters that are unknown or have been ignored due to their relative unimportance and idealisation resulting from mathematical expressions. The effects of these uncertainties could be so negligible that the resulting reliability calculation will be unaffected. Additional investment to improve the model would therefore not be justifiable in a risk-based framework. The process of evaluating model uncertainty is accepted as being one of the more difficult parts of structural reliability analysis as the model uncertainty is dependent on many factors and information about the uncertainty is scarce and fragmentary (Ditlevsen and Madsen, 1996). The process of estimating model uncertainty has also been addressed with GLUE (described in Section 3.2.4) which addresses the same problem in a different context.

Ditlevsen (1982) proposed a method of evaluating uncertainty in structural reliability that does not use model outputs or processed data that can be used by experts in making a judgement of model uncertainty. The uncertainty is represented by randomly deforming the idealised limit state function into a 'possibly true' limit state surface. The failure surface $g(\underline{x})=0$ becomes $g(\underline{V}(\underline{x}))=0$ where $\underline{V}(\underline{x})$ is a random vector field, this is shown in Figure 3.12.

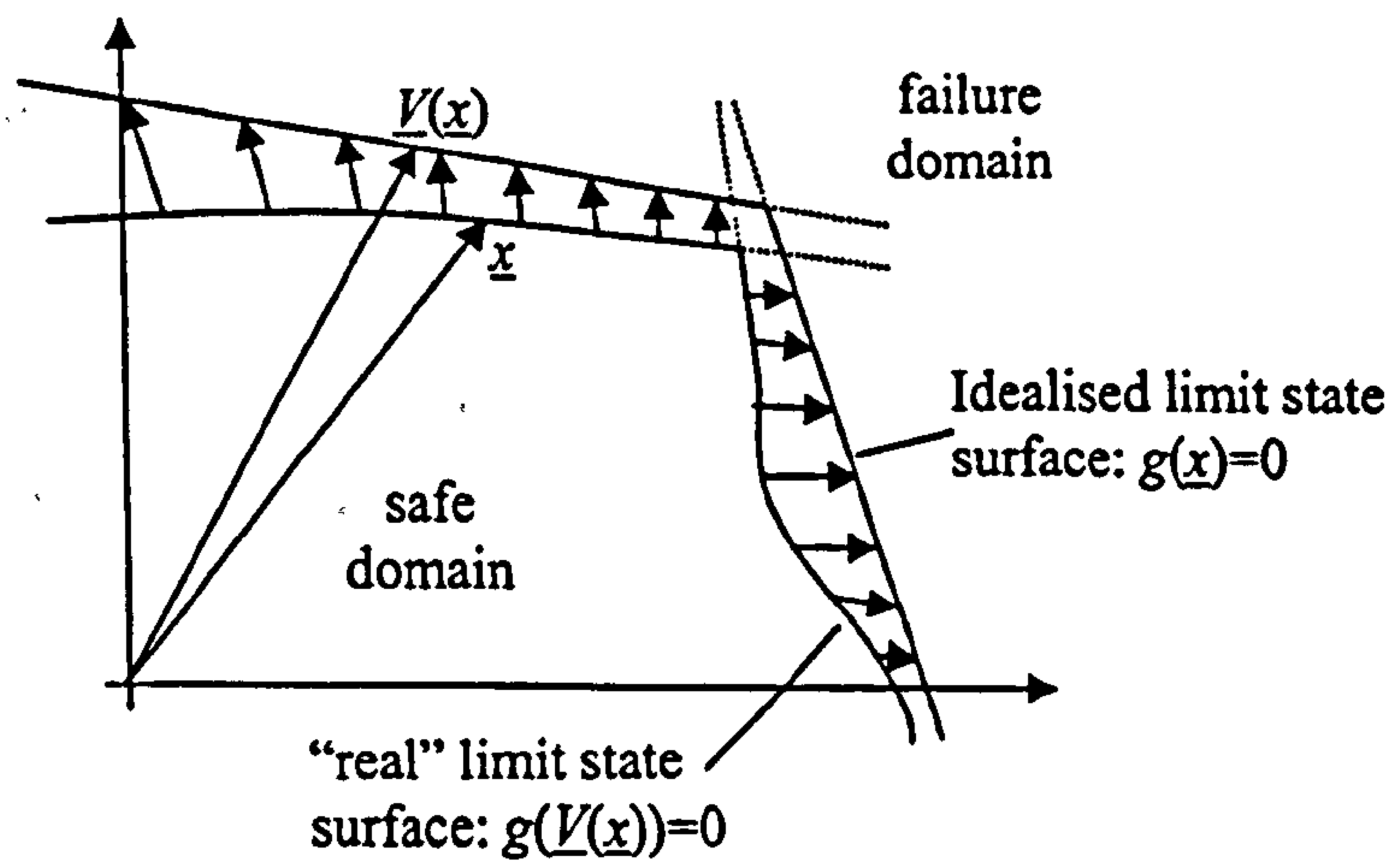

Figure 3.12 A randomly distorted limit state surface resulting from defining a random vector (Ditlevsen, 1982)

The reliability is therefore the probability that the random vector $\underline{V}(\underline{X})$ has not failed. The simplest example of the field $\underline{V}(\underline{x})$ is:

$$\underline{V}(\underline{x}) = \underline{x} + \underline{J} \qquad (3.58)$$

where J is a random vector independent of $\underline{x}$. A more flexible and less idealised representation is obtained using:

$$\underline{V}(\underline{x}) = \underline{H}\,\underline{x} + \underline{J} \tag{3.59}$$

where $\underline{H}$ is a random matrix independent of $\underline{x}$. The distribution of the input random vector $\underline{X}$ is therefore replaced with the probability distribution of $\underline{V}(\underline{x})$ in the reliability analysis. The model uncertainty is therefore accounted for by modifying the distributional properties of the input vector (Ditlevsen and Madsen, 1996).

3.4.3. System failure probability

A complex system may contain a combination of parallel and series sub-systems. The failure probability for a series system is governed by the component with the highest failure probability. The failure, F, of a component of a series system of n members will have a failure probability, Pf, defined as:

$$P_f = P\,(F_1 \cup F_2 \cup \ldots \cup F_n) \tag{3.60}$$

This is shown by the failure space in Figure 3.13, and equates to convolution of the joint probability distributions in this space. Parallel systems fail when a combination of system components fails. The failure probability of a parallel system is given by Equation 3.61.

$$P_f = P\,(F_1 \cap F_2 \cap \ldots \cap F_n) \tag{3.61}$$

This is represented in Figure 3.13 by the intersecting failure space. As with series systems, this can be calculated by integrating the limit state functions over the failure space. Equations 3.60 and 3.61 clearly provide two limiting conditions on the probability of failure of any system. However, the assumptions of completely serial or parallel systems is often difficult to justify and can provide unhelpfully wide bounds on the probability of failure. Real systems are likely to contain a network of elements that demonstrate varying degrees of parallel and serial behaviour.

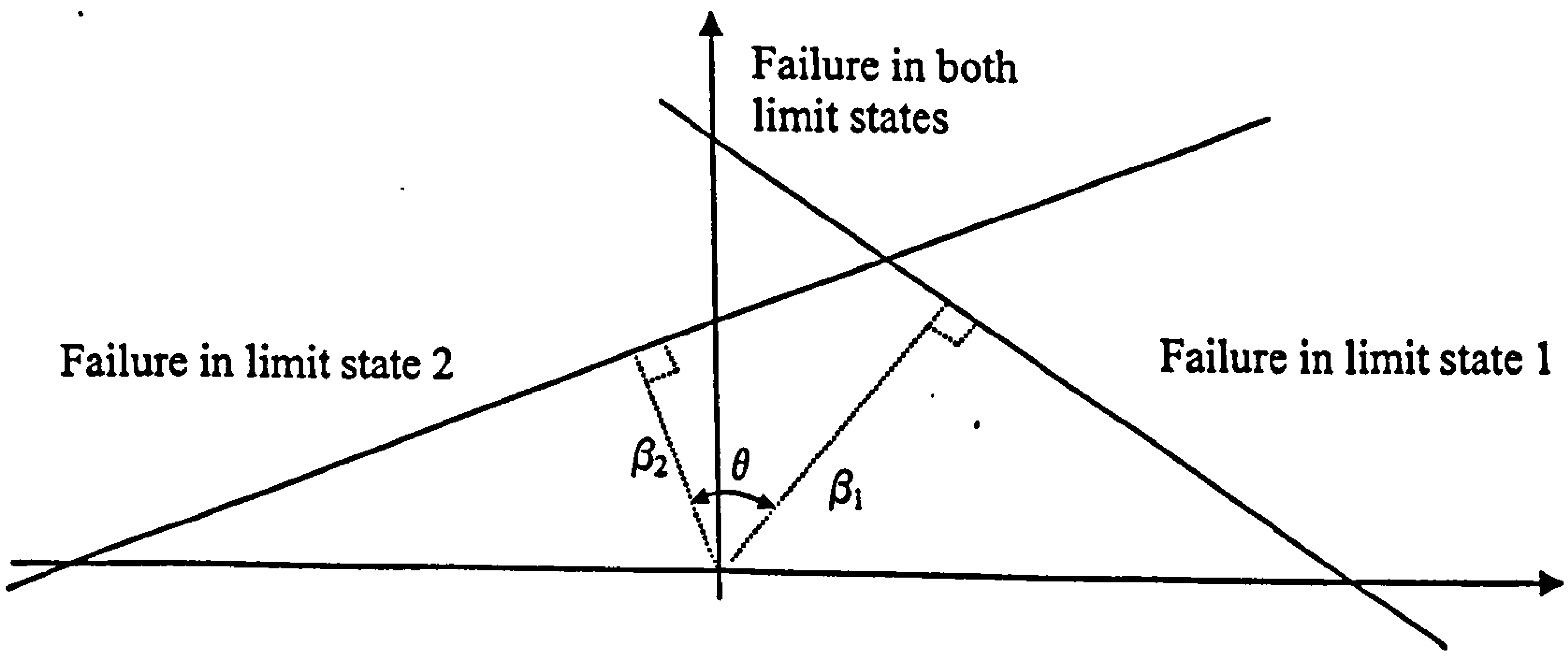

Figure 3.13 The failure space of a two-component system

Correlation

Failure modes and defences demonstrate a degree of correlation if some or all of their resistance and load variables are shared. The correlation can result from dependency of loadings, for example, a storm surge along a stretch of coastline, or dependency between system elements such as the geotechnical properties of a river bank.

The coefficient of correlation is a dimensionless value between zero and one, which correspond to uncorrelated and fully correlated coefficients respectively. The coefficient of correlation, ρ, between the reliability, G, of two defences, i and j, with resistance, R, being subject to a load S, is (CUR and TAW, 1990):

$$\rho(G_i,G_j)=\frac{\rho(R_i,R_j).\sigma(R_i).\sigma(R_j)+\rho(S_i,S_j).\sigma(S_i).\sigma(S_j)}{\sigma(G_i)\sigma(G_j)} \tag{3.62}$$

For correlation coefficients greater than 0.7, Ditlevsen second-order bounds (introduced later in this section) can be rather wide (Thoft-Christensen and Baker, 1982).

In the context of flood defences, the coefficient of correlation may be a function of the distance between two defences and over long distances the autocorrelation will tend to zero as the actual parameters at two locations that are far apart will have no relationship. This is usually the case for breaching where resistance parameters are variable, but for overtopping the storm level can correlate the defences over a long distance. CUR and TAW (1990) recommend that the correlation length (length over which the correlation tends to zero) is taken as 500m. When implementing a first order reliability analysis, the Rosenblatt transformation (described in Section 3.4.2) is used to transform arbitrarily distributed parameters into uncorrelated normally distributed parameters.

It is useful to calculate the overall failure probability of a structural system rather than just one of its failure modes. Calculating correlation coefficients of defence resistance is rarely possible in England and Wales due to insufficient data (although it may be possible to correlate loadings along a length of defences). The example in Figure 3.13 shows the failure space for a system with two limit states, the review of flood defence failure in Appendix D shows that defence can have many more failure modes than this. A single flood defence is also part of a larger system of defences, increasing the dimensionality of the system significantly. Exact determination of the failure probability of complex systems is not always possible and a numerical solution is often time-consuming (Thoft-Christensen and Baker, 1982) and so it is usually more useful to estimate bounds on the failure probability.

First-order bounds

First order failure bounds, so called because they consider only single element failure, assume perfect dependence and perfect independence between the system's failure modes. For independent failure modes, the failure probability is the product of one minus the failure probabilities (Equation 3.63). For dependent failure modes the weakest failure mode will always be less likely to fail. Hence bounds on the failure probability, P_f, of series systems of n components can be calculated from:

$$\max_{i=1}^{n}(P_{f_i}) \le P_f \le \prod_{i=1}^{n}(1-P_{f_i}) \tag{3.63}$$

For parallel systems, these bounds are given by:

$$\prod_{i=1}^{n}(P_{f_i}) \le P_f \le \min_{i=1}^{n}(P_{f_i}) \tag{3.64}$$

Second-order bounds

For many systems, first order bounds are too wide to be useful (Throft-Christensen and Baker, 1982) however, Ditlevsen (1979b) proposed higher order, narrower bounds. Second order bounds consider the joint failure probabilities of two system elements as well as individual element failure as shown in Equation 3.65.

$$Pf_1 + \sum_{i=2}^{n}\left\{\left[Pf_i - \sum_{j=1}^{i-1} Pf_i \cap Pf_j\right], 0\right\} \le Pf \le \sum_{i=2}^{n} Pf_i - \sum_{i=2}^{n} \max_{j<i}\left[Pf_i \cap Pf_j\right] \tag{3.65}$$

3.4.4. Reliability assessment of flood and coastal defence structures

It is the need to make a quantitative assessment of flood risk that is driving the need to estimate failure probabilities. These values can be incorporated into a systems analysis to incorporate multiple failure modes and structures. However, it is important to recognise that the accuracy of probabilities for singular events (events that have never been observed and may never be observed) is often questioned (Bedford and Cooke, 2001) and cannot be falsified. Under these circumstances the failure probability should be thought of as a propensity interpretation of probability, meaning it is a measure of the tendency for the event to occur (Popper, 1959, Bunge, 1981).

First order second moment methods have a sound grounding in the field of coastal defence having been successfully applied on numerous occasions (Burcharth, 1992, Burcharth and Sorensen, 1998, Hussaarts *et al.*, 2000, Kortenhaus *et al.*, 2002, Voortman *et al.*, 2002 to list a few), and these applications are clearly transferable to fluvial defences. However, application of the reliability methods themselves can add an extra degree of uncertainty. Level II methods can result in errors from evaluating the failure probability caused by linearization of the failure function. Point estimate methods (Rosenblueth, 1975, 1981) have been successfully applied in coastal flood risk assessment by Reeve (1998, 2002). Harr (1995) has formalised their inclusion within a reliability analysis to estimate failure probabilities. However, as with first order second moment methods, functions with a high degree of nonlinearity produce significantly erroneous results if the higher moments are unknown (Li, 1992). Whilst more accurate (but processing intensive) level III methods can be applied, ultimately a reliability analysis is as accurate as the failure model and data used.

Guidance on applying reliability methods in flood and coastal defence has focused on the design of structures rather than their assessment (CUR and TAW, 1990 and Oumeraci *et al.*, 2001). The large amount of data required to perform a reliability analysis limits its use as an assessment tool in situations of sparse or incomplete data. Despite the fact that probability distributions are well established to represent uncertainty in wave and water levels (CIRIA and CUR, 1991 and CEHW, 1999) and there is much well documented research and guidance, there has traditionally been a resistance from engineers to apply reliability methods in the design of flood and coastal defences where reliability methods are still relatively new and design and investment decisions have to be made under extreme uncertainty. Despite this, probabilistic methods offer many advantages:

- they offer a consistent and transparent approach to design and assessment,
- design costs can be optimised and investment decisions made more efficiently,
- they can account for the uncertainty in the design parameters, and,
- they allow a quantitative assessment of flood risk to be made to monitor system performance.

Model uncertainty parameters, based on the method of Ditlevsen (1982), have been used in the reliability analysis of coastal structures with several limit state functions. Burcharth (1992) gives an example application using Hudson's rock armour stability equation and defining the model uncertainty, A, as A~N(1.0,0.18). These values are usually based on the scatter in experimental data that was used to generate the original failure function. The use of such parameters to capture model uncertainty is often controversial as the empirical evidence supporting these approaches is questionable (Blockley, 1999) and more significant are the uncertainties stemming from transferring a parametric model developed from experimental data to a specific site (Hall, 2003). The second order approach of Burmaster and Wilson (1996) requires uncertainty to be mapped onto a probability distribution for each parameter. However, as described previously in Section 3.2.4, this uncertainty is not best represented by probability distributions. Moreover, the dependency relationship between marginal distributions of model output and model dependability is seldom clear (Hall, 1999). For the purposes of structural design, model uncertainty is usually accounted for in a more deterministic manner using partial safety factors (Oumeraci *et al.*, 2001).

Caution is also required when using certain limit state formulae in a reliability analysis as they are usually based on scale model tests that often show considerable scatter in the results. To address the uncertainty in scale model results, researchers have recommended "safe" values of model coefficients. Whilst these provide a sound basis for design, their inherent conservatism means that they will result in biased risk assessments. To avoid this bias, the original experimental data may need to be revisited to estimate "most likely" rather than "safe" values of model coefficients together with an estimate of the surrounding uncertainty. For some well know formulae this has already been done using probability distributions (Stoutesdijk *et al.*, 1998, Bezuijen and Kruse, 1998). Hall (2003) demonstrated that eliciting expert judgement to generate fuzzy membership functions can be a useful method of representing the uncertainty associated with these parameters and the transferring of a model derived from experimental data to a specific site.

Lindley (1987) argues that probability "*is the only sensible description of uncertainty and is therefore adequate for all problems involving uncertainty*" and HR Wallingford (2002) note that many natural phenomena conform well to probability distributions. However, much of the scepticism related to the application of reliability methods stems from their inability to incorporate evidence on the dependability of input parameters and evidence from the diverse range of sources with different degrees of fuzziness that is the case in decision-making in flood and coastal defence. It is rare in the UK that there is enough information to be able to generate accurate values for the moments of parameters that control failure mechanisms. Other parameters can not accurately be defined by a point value. Membership functions which were introduced in Section 3.2.4 offer a useful alternative of capturing expert judgement which is based upon possibilistic rather than probabilistic reasoning. In situations where expert judgement cannot provide a meaningful probability distribution, a possibility distribution of values can be estimated. In its simplest form, this places bounds on the possible value of a system parameter, for example rock diameter. A more informative description would be a fuzzy set. The use of possibility descriptions means that probabilities do not have to be estimated for each possible value. Uncertainty associated with the use of possibility distributions can be propagated through the reliability analysis and reflected in the final estimate of failure probability. Whilst this appears to increase the uncertainty associated with the reliability assessment, in reality, the decision-maker is better informed about where the uncertainties in the data lie. Estimation of a parameter distribution is itself a form of uncertainty.

Whilst a Bayesian may argue that probability distributions should be estimated initially and improved with time, a lack of data, infrequent failure information and irregular monitoring mean this approach is often inappropriate in flood defence. Because of a lack of training data, the use of GLUE which is undoubtedly a powerful methodology for estimating model uncertainty, is limited for estimating uncertainty in rare (such as an extreme flood) or unrepeatable (such as a hydrologically activated landslide) events (Hall and Anderson, 2002). This is the case for reliability analyses. Flood defence failures are not frequent events and those that have occurred have often not been recorded adequately. This, coupled with the fact that failure mechanisms are often affected by site specific factors means that methods such as GLUE that rely on training data will not be appropriate. GLUE is also not without the need for subjective judgements, such as the likelihood measures, that are used in other model uncertainty methods, however, it is argued that by making these judgements explicit they can at least be subjected to peer review and scrutiny.

3.5. RECEPTOR - IMPACT ESTIMATION

The receptor "*refers to the asset that may be harmed*" (ICE, 2001). To be able to estimate flood risk an assessment of the impacts of flooding is required. This section describes the methods used to estimate potential flood consequences.

If there were no development in floodplains there would be no flood risk. However, the benefits offered from living near rivers or coastlines and from farming the fertile floodplain land have resulted in towns and cities growing in these areas. In the Netherlands and parts of East Anglia, land has been reclaimed from the sea creating areas of vulnerable low-lying land. Compounded by sea level rise and an increasing population the impacts of flooding are likely to increase in the future. Flood risk can therefore be reduced by receptor management. In the United States, this has been taken to one extreme where entire communities have been relocated to reduce flood risk (Miletti, 1999). In England and Wales controlling future development in floodplains is now recognised as being an important part of flood risk management (ICE, 2001). Receptor management is not always considered to be a suitable option. For example, in the Netherlands where the majority of the country is at risk from flooding the pathway is the dominant method used to control flood risk.

Flood impacts are categorised based on whether they are direct or indirect and tangible or intangible. These can be further classified based on whether they are primary or secondary losses as shown in Table 3.2 (Smith and Ward, 1998).

Table 3.2 Categories of flood loss potential (Smith and Ward, 1998)

FLOOD LOSSES							
DIRECT				INDIRECT			
TANGIBLE		INTANGIBLE		TANGIBLE		INTANGIBLE	
Primary	*Secondary*	*Primary*	*Secondary*	*Primary*	*Secondary*	*Primary*	*Secondary*
Physical damage to property	Costs of complete restoration	Loss of human life	Ill health of flood victims	Disruption of traffic and trade	Reduced spending power in community	Increased hazard vulnerability of survivors	Out-migration and reduced confidence in area

Direct tangible losses

Traditionally flood risk has been expressed in economic terms and current assessments of flood risk clearly emphasise this, although there is increasing recognition of the need to assess other risks (DEFRA, 2000). The Flood Hazard Research Centre (FHRC) at Middlesex University has studied in detail the effects of direct tangible damage (Penning-Rowsell and Chatterton (1977), Middlesex University (1990) and Penning-Rowsell *et al.* (2002)) which resulted in the publishing of depth-damage curves such as those in Figure 3.14 (where a negative depth indicates basement flooding). Primary losses in urban areas results from direct physical damage to property. Primary losses in rural areas are a result of crop damage or loss of livestock and agricultural infrastructure (*eg.*

irrigation and drainage systems). Agricultural damages vary according to the type of crop and season. A previous national assessment of flood risk in England and Wales has shown that direct tangible losses are usually dominated by urban damage. Halcrow *et al.* (2001) calculated the average annual economic damage to be £814million and of this 12% was due to loss of agriculture and 0.5% due to traffic disruption.

Secondary direct tangible losses relate to restoration of buildings. After restoration a loss in property value resulting from the flood may also be expected, but Montz (1992) has shown that house prices should be expected to recover and that flood risk was not an important long-term consideration in buying property. However, there is evidence to suggest that as insurance companies increase their premiums after major floods, or refuse to insure homeowners in floodplains, the property value is significantly reduced (ICE, 2001).

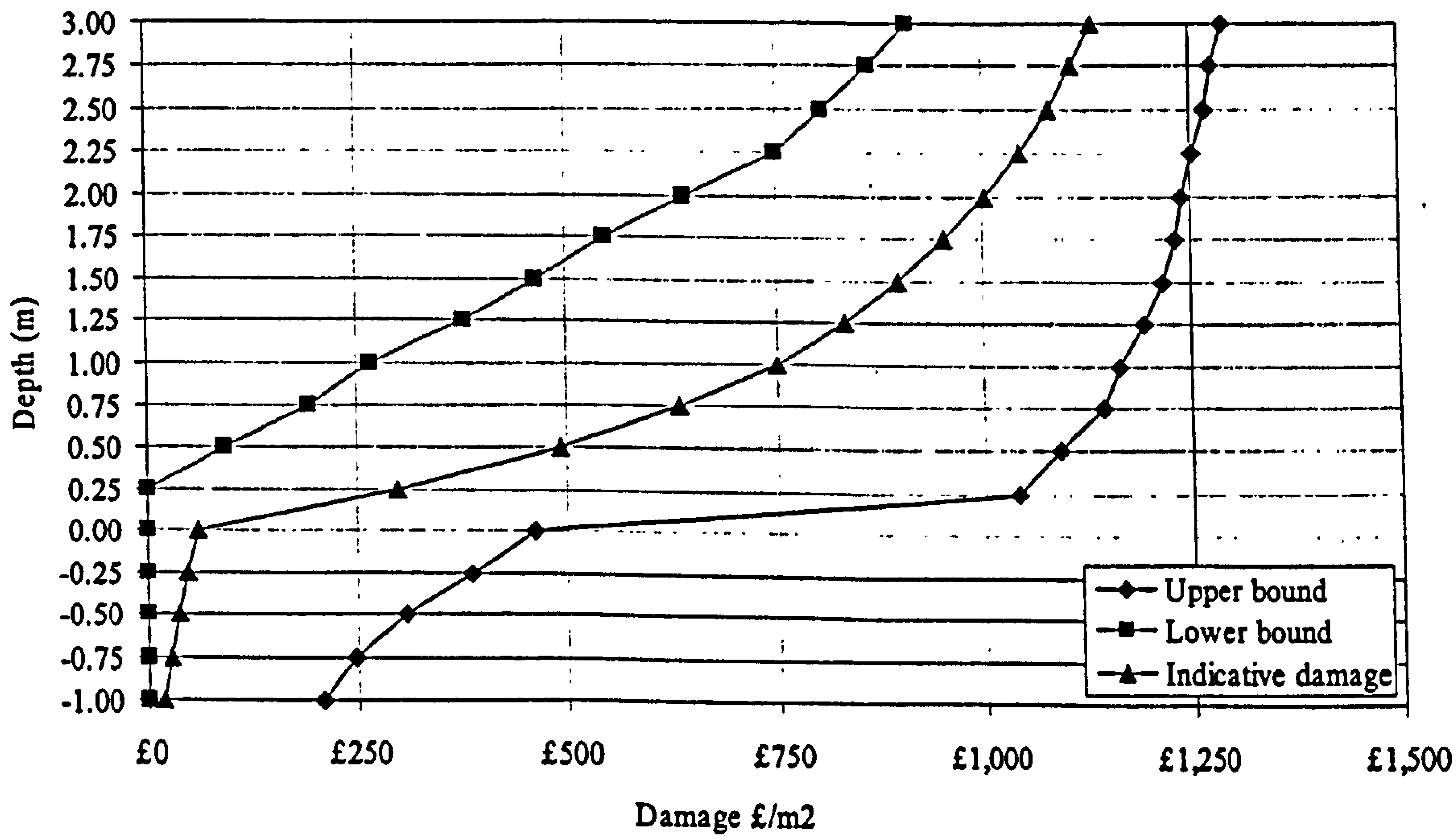

Figure 3.14 An example of a depth-damage curve for "Retail shops", showing lower, upper and indicative damage bounds (Middlesex University, 2002).

Direct intangible losses

Primary direct intangible losses are the mortality rates associated with a flood event. These can be deaths attributed to drowning or as a result of water-borne disease or starvation. Guidelines for levels of acceptable individual risk are given by the HSE (1989). Methods for estimating the number of deaths from flooding (and other disasters) have been proposed by Vrijling *et al.* (1995). This approach calculates both individual risk, *IR*, which reflects the risk to the individual at a given location.

$$IR = P_f \times P_{d|f} \tag{3.66}$$

where P_f is the probability of a flood and $P_{d/f}$ is the probability of death given the flood. This is used to limit the risks of nearby hazards and transport routes (Jonkman *et al.*, 2002). A national level of risk, or societal risk, *SR*, is also measured:

$$SR = E(N) + k.\sigma(N) \qquad (3.67)$$

where $E(N)$ represents the expected number of deaths which is increased by the risk aversion index k (which represents the public attitude towards large disasters and different types of risks) of the standard deviation.

Secondary losses are related to physical or mental ill-health resulting from the flooding. Physical illness can come as a result of water-borne disease or polluted water. Long term distress and in some circumstances mental illness can result from loss of family or possessions and general disruption. Hoque *et al.* (1993) showed the devastating effect of the 1991 floods on public health in Bangladesh. Prediction of the severity of these impacts is difficult. Even after a flood, this assessment is challenging. Typically, such assessments involve a comparison of pre- and post-flood event visits to doctors or hospitals, other effects such as depression or stress are based on interview and questionnaire studies (Bennet, 1970). However, these require a good knowledge of the pre- event situation and are naturally subjective.

Recently measures of social vulnerability (described in more detail in Chapter 2) have been suggested for England and Wales to provide an indication of the likely seriousness of the direct intangible effects of flooding (DTLR, 2000 and Tapsell *et al.*, 2001). Intangible losses also result from a drop in lifestyle quality. Penning-Rowsell *et al.* (1992) have suggested methods for evaluating the losses associated with recreation and environment in the coastal environment. Guidance for evaluating the cost of the environment are described in more detail in Chapter 2 and DEFRA (2000c).

Indirect tangible losses

Whilst often quantifiable, less research has been done on measuring indirect tangible damages compared to direct tangible damages, although Parker *et al.* (1987), the Environment Agency (1996) and the Department of Transport (2001) have proposed methodologies for measuring losses caused by disruption to the transportation infrastructure. Penning-Rowsell and Parker (1987) identified that these losses can extend beyond the flooded area. For example, disruption of the transport infrastructure results in diversions thereby wasting people's time and fuel, and possibly disrupting local trade. Other indirect tangible costs include costs of the emergency response services and loss of public utilities such as gas, electricity or water. Secondary losses are caused by long term losses in trade, interconnection of industries within the same flooding region or a change in the spending priorities of consumers as a result of the flooding and for large floods these damages can be substantial (Olsen *et al.*, 1998). Recent flooding in England and Wales has made some areas of the floodplain likely to be left uninsured resulting in a dramatic drop in house price in these areas (ICE, 2001).

Indirect intangible losses

Indirect intangible losses are the hardest to measure and it is difficult to differentiate between primary and secondary losses (Smith and Ward, 1998). Types of intangible losses include long term environmental changes, changes in social behaviour and population migration of those who are able to leave thereby resulting in greater vulnerability of the remaining population.

3.6. SUMMARY

Using the source-pathway-receptor framework the key stages of risk assessment have been reviewed. This has been complimented by a review of uncertainty which is present at each stage of the risk assessment. The key needs for flood defence managers identified in Chapter 2 required the research to place a greater emphasis upon assessing failure probabilities of defences and flood defence systems. The analysis in this chapter therefore focussed on methods available to assess the reliability of flood defence systems and techniques available to handle the uncertainty associated with the data needed for these analyses.

Two of the key needs for flood defence managers identified in Chapter 2 were:

(i) a quantitative systems-based flood risk assessment methodology, and,

(ii) a method to describe defence condition probabilistically.

A flood risk assessment requires an assessment of loads, likelihood of inundation and consequences of inundation. Each part of this assessment has associated uncertainties. These should be kept separate and only combined when calculating the final risk to allow the decision-maker to identify the main sources of uncertainty. The level of acceptable uncertainty will be dependent on the decision and available resources. A more detailed analysis, whilst reducing uncertainty, will also require more resources and is therefore inappropriate for a minor investment decision. A tiered risk assessment methodology that uses a level of analysis appropriate to the decision being informed is therefore proposed. The three tiers for this methodology are outlined in Chapter 4. A method suitable for application on a national scale is described in detail.

A review of techniques available to flood defence managers has identified reliability methods as being a useful and well established tool for assessing failure probabilities of flood defences. However, their application so far has involved the use of evidence expressed as either deterministic or probabilistic values. Much information pertaining to flood defence failure is suited to being expressed in these formats (*eg.* distributions of water levels). However, expert judgement which has a pivotal role in flood defence management is much better expressed as a membership function. Membership functions can also be used to capture uncertainty associated with incomplete or scarce data. Again, this is frequently the case for the flood risk manager. Reliability analyses need to be able to incorporate this information, placing bounds on the failure probability which represent the uncertainty in the evidence. This enables quantitative bounds on the probability of defence failure to be estimated using the best available information without pre-supposing any parameter distributions. Upper and lower failure bounds for a defence can be calculated using ordinary system bounds or Ditlevsen bounds. A novel approach to condition characterisation of flood and coastal defences that incorporates membership functions within a reliability analysis of flood and coastal defences is described in Chapter 5. This methodology corresponds to the detailed level of the tiered risk assessment described in Chapter 4.

Whilst a probabilistic risk assessment and reliability analyses of defences provide useful indicators of system and defence performance, other evidence of performance should be included in order to broaden the perspective of the decision-maker. A decision-support tool that enables this is described in Chapter 6.

Chapter 4

A tiered approach to flood risk assessment

4.1. OVERVIEW

The review of the current state of flood defence management in Chapter 2 identified the need for a flood risk assessment that provides a rational basis for development of flood management policy, allocation of resources and monitoring the performance of flood mitigation activities. In order to support a wide range of management decisions a tiered framework has been developed that builds on the work of Meadowcroft *et al.* (1996). This chapter provides an overview of three increasingly detailed levels of risk assessment and describes in detail a method appropriate for making a national-scale flood risk assessment. The intermediate level of assessment is described completely and the detailed level, which is at an earlier stage of development, is briefly outlined.

A national-scale risk assessment presents particular challenges in terms of data acquisition and manipulation, numerical computation and presentation of results. A methodology that addresses these difficulties through appropriate approximations has been developed and applied in England and Wales. The methodology represents the flood defence system in sufficient detail to test alternative policy options for macro-scale investment decisions in flood management. Defence condition is assessed against proneness to overtopping[•] and breaching using curves constructed by experts that describe the conditional failure probability of a defence. Appropriate assumptions about defence system behaviour are made. Flood outlines and depths are generated using a rapid parametric inundation routine. Potential economic and social impacts of flooding are assessed using national databases of floodplain properties and demography. A case study of the river Parrett catchment and adjoining sea defences in Bridgwater Bay demonstrates the application of the method and presentation of the results in a Geographical Information System.

4.2. A TIERED RISK ASSESSMENT METHODOLOGY

As described in Chapter 2, considerable numbers of people and property are at threat from flooding and recent flooding has demonstrated the need for improved management of flood defences. Flood risk assessment is required to support the appraisal of policy options, resource allocation and

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[•] In this case, overtopping refers to overtopping dominated by overflow for fluvial defences and overtopping dominated by wave action for coastal defences. This is explained in more detail later in the Chapter.

as a measure of performance of the substantial annual investment in flood management (HM Treasury, 2002).

The amount of resources invested in data acquisition and analysis for risk assessment should reflect the importance of the decision(s) that are being informed. Flood management takes place at a number of levels, ranging from national policy decisions to planning decisions in a river catchment or coastal cell down to individual scheme design and day to day operational decisions. A flood risk assessment with three tiers is proposed. Each tier provides a progressively more thorough and accurate assessment of flood risk appropriate to the decision being taken (Table 4.1). In order for the *High Level* analysis to be performed on a national scale, only data that is available for the entirety of England and Wales can be used. The *Intermediate Level* incorporates additional information on loading, floodplain topography and defence structure. The *Detailed Level* uses information about the autocorrelation of loads and composition of the defences and a much more detailed study of their proneness to failure to provide the most accurate flood risk assessment.

Table 4.1 Hierarchy of risk assessment levels, the decisions they inform, the data needed and methodologies used to implement them

Level	Decisions to inform	Data sources	Methodologies
High	• National assessment of economic risk, risk to life or environmental risk • Prioritisation of expenditure • Regional planning • Flood warning planning	• Defence type • Condition grade • Standard of Protection • Indicative flood plain maps • Socio-economic data • Land use mapping	• Generic probabilities of defence failure based on condition assessment and crest freeboard • Assumed dependency between defence sections • Empirical methods to determine likely flood extent
Intermediate	*Above plus:* • Flood defence strategy planning • Regulation of development • Maintenance management • Planning of flood warning	*Above plus:* • Defence crest level and other dimensions where available • Joint probability load distributions • Flood plain topography • Detailed socio-economic data	• Probabilities of defence failure from reliability analysis • Systems reliability analysis using joint loading conditions • Modelling of limited number of inundation scenarios
Detailed	*Above plus:* • Scheme appraisal and optimisation	*Above plus:* • All parameters required describing defence strength • Synthetic time series of loading conditions	• Simulation-based reliability analysis of system • Simulation modelling of inundation

The three tiers of flood risk assessment share common elements. Each level requires:

- a description of loadings (water levels and wave heights),
- an estimation of defence failure probabilities,
- an inundation model, and,
- an estimation of consequences (people and property in the flooded area).

The methods used to achieve these aims vary according to the required detail of analysis.

4.3. HIGH LEVEL METHODOLOGY

4.3.1. Overview

To calculate a national assessment of flood risk, it is necessary to aggregate the average annual damage (AAD) over all the floodplains in England and Wales. An overview of the method used is described in this section and shown by the flowchart in Figure 4.1, a more detailed description is provided in Sections 4.3.2 to 4.3.5 and results from a case study are shown in Section 4.4.

The only nationally available information on the potential extent of flood inundation are the Indicative Floodplain Maps (IFMs) which are outlines of the area that could potentially be flooded in the absence of defences in a 1:100 year return period flood for fluvial floodplains and a 1:200 year return period flood for coastal floodplains. The IFM was derived from a variety of sources: topographic maps, flood models and/or historical flood records. For the purposes of the risk assessment the Indicative Floodplain is divided into flood damage zones not greater than 1km × 1km. Each flood damage zone is associated with a system of flood defences which, if one or more of them were to fail, would result in some inundation of that zone. Defences can be associated with more than one flood damage zone.

The probability of failure of a flood defence system can be estimated using the methods of structural reliability analysis (CUR and TAW, 1990 and Melchers, 1999). However, these methods require probability distributions for the hydraulic loads and parameters describing defence response as well as analytical or numerical expressions for each failure mode. Unfortunately in England and Wales there is no nationally available probabilistic loading information. Moreover, information on defence crest level and structural strength parameters is not consistently available. Clearly it is crucial to be able to establish a relationship between loading and crest level. In England and Wales the only information nationally available is a measure of the Standard of Protection (SOP), which is an assessment of the return period at which the defence will significantly be overtopped or overflowed. In the high level methodology the SOP is used as a benchmark for a generic load distribution. A generic probability distribution of defence failure, conditional on load, defence type and condition, is used to estimate the probability of failure by two mechanisms, overtopping and breaching. An estimate is made of the probability of every combination of defence failure within a flood defence system, taking into account the dependency between defence sections.

For each combination of defence failure an approximate flood outline is generated using parametric routines that estimate discharge through or over the defence and inundation characteristics of the floodplain. Whilst high resolution topographic data is becoming available on a national basis in the UK, without information on flood water levels it cannot be used to estimate depths of inundation. Estimation of flood depth therefore has to be based on statistical data from real and simulated floods in a range of floodplain types and floods of differing severity. Economic risk is calculated based on damage to properties and agricultural land use within the flooded area. The population at risk is derived from census data and a measure of the social harm of flooding is obtained from Social Flood Vulnerability Indices (Tapsell *et al.*, 2002).

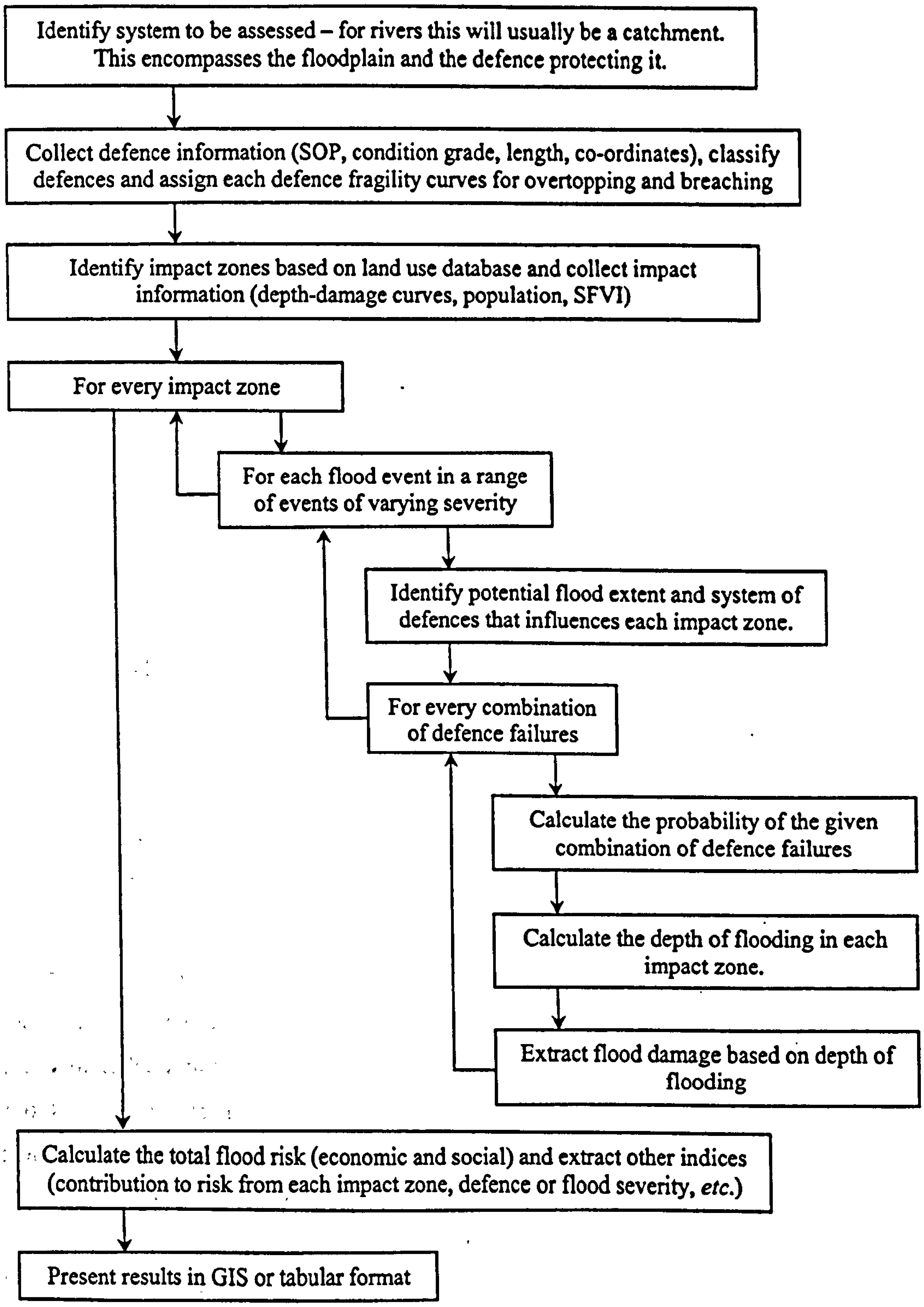


Figure 4.1 Overview of the high level flood risk assessment methodology

4.3.2. High level systems reliability analysis

Figure 4.2 shows a simple example of a flood defence system. The floodplain (defined by the limits of the IFM) is divided into m flood damage zones, labelled $z_1, z_2, \ldots, z_m$, each 1km × 1km in size. Breaching or overtopping of one of n flood defences, labelled $d_1, d_2, \ldots, d_n$, results in inundation of one or more of these flood damage zones. For each damage zone it is necessary to calculate the probability of every combination of defence failure that may cause flooding in that zone. Defining the failure of defence d_i as D_i and non-failure as $\overline{D_i}$, a damage zone protected by two defence, d_1, d_2, has three possible defence failure scenarios $D_1 \cap D_2$, $D_1 \cap \overline{D_2}$, $\overline{D_1} \cap D_2$, and one non-failure scenario, $\overline{D_1} \cap \overline{D_2}$.

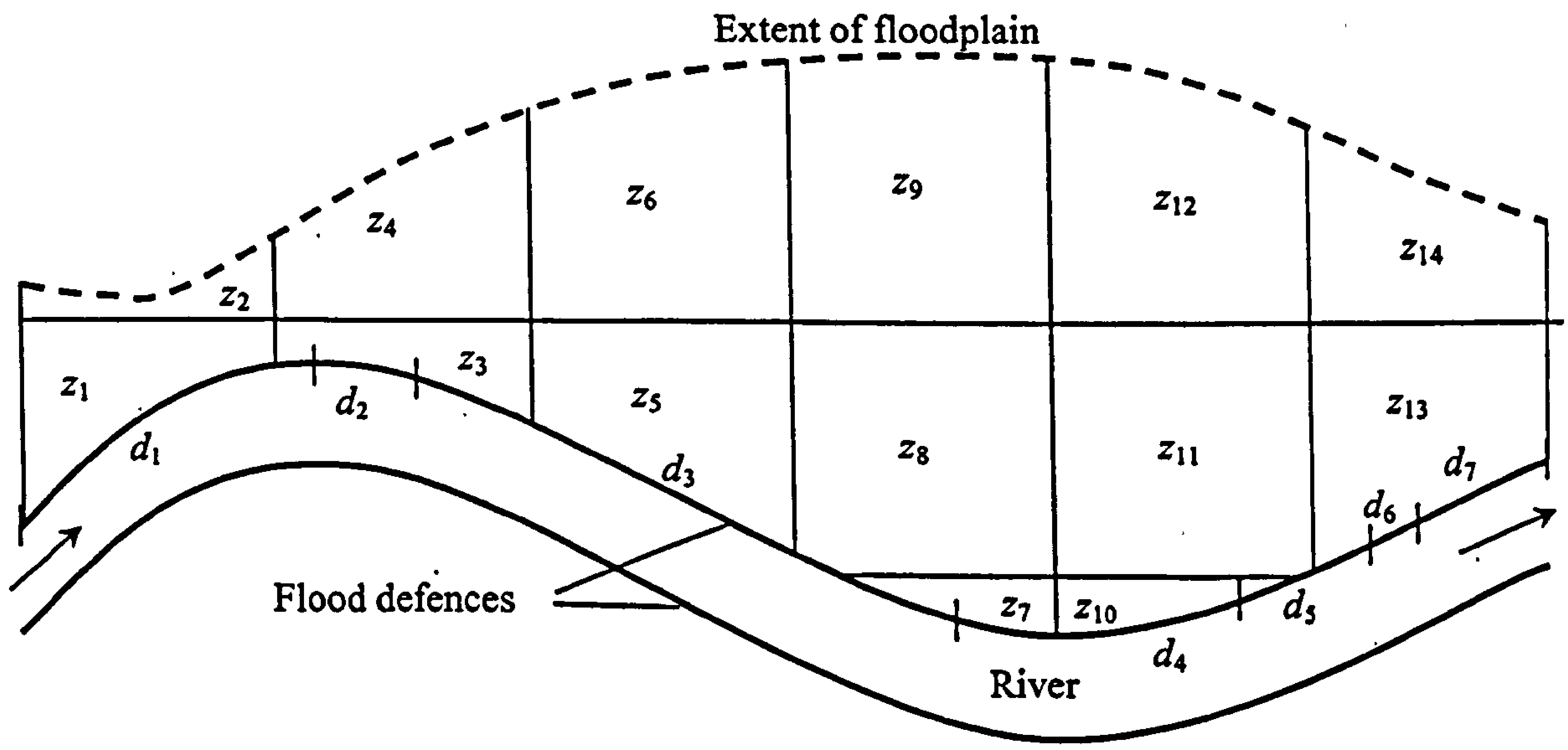


Figure 4.2 A floodplain that has been divided into impact zones (z_1-z_{14}) and protected by flood defences (d_1-d_7)

The probability of failure of a defence is a function of the probability of an extreme load (river water level or combined wave height and sea level) and the defence's resistance to that load. In the absence of information on the frequency of extreme loads a factor x multiplied by the SOP has been adopted as a proxy for load. Thus if the SOP is 1:100 years, then a load with $x = 0.5$ corresponds to a 1:50 year return period event. The annual probability $P(X \geq x)$ of an event X with severity greater than or equal to x at a defence with Standard of Protection SOP is given by:

$$P(X \geq x) = 1 - \left(1 - \frac{1}{s.x.\mathrm{SoP}}\right)^s \tag{4.1}$$

where s is the number of load events in a year. For large SOP Equation 4.1 can be approximated as:

$$P(X \geq x) = \frac{1}{x\mathrm{SoP}} \tag{4.2}$$

Each defence section d_i is assigned, on the basis of its defence classification and condition, a conditional probability of failure given a load x, $P(D_i|x)$, for a range of values of x. In reliability analysis a conditional probability distribution of this type is referred to as a 'fragility curve' (Casciati and Faravelli, 1991 and Chapter 5). Typical fragility curves are illustrated in Figure 4.4 and Figure 4.5. The fragility curve can be combined with the loading distribution to generate an unconditional probability of defence failure, $P(D_i)$:

$$P(D_i) = \int_0^\infty p(x)P(D_i \mid x)dx \tag{4.3}$$

where $p(x)$ is the probability density function of the load x. The product x.SOP is a measure of the severity of the hydraulic load, so it is replaced by the symbol l, in which case:

$$P(D_i) = \int_0^\infty p(l)P(D_i \mid l)dl \tag{4.4}$$

and $P(D_i|l)$ can be obtained from the fragility curve by reading off at $x = l$/SOP. For the purpose of the current analysis the fragility curve is defined in discrete terms at q levels of l: $l_1,\ldots, l_q$, enabling Equation 4.4 to be re-written as:

$$P(D_i) = \sum_{j=2}^{q-1}\left[P\left(L \geq \frac{l_j + l_{j+1}}{2}\right) - P\left(L > \frac{l_j + l_{j-1}}{2}\right)\right]P(D_i \mid l_j) \tag{4.5}$$

where L is a random variable representing the hydraulic load. Unlike the commonly used First Order Reliability Method (Melchers, 1999) this discrete approach allows arbitrarily shaped distributions of load and structural response. From a computational point of view, the discrete approach is attractive because it generates exact bounds on the probability of failure, illustrating numerical errors in the same format as the other uncertainties in the analysis.

To estimate the probability of occurrence of a scenario in which a given number of defences in a system fail requires information about the dependency between the variables describing system behaviour. In this national-scale analysis three simple assumptions are made:

(1) Loading of all defences in a defence system is considered to be fully dependent *i.e.* all defences are subject to the same load at the same time. The relief of load on downstream defences due to failure of an upstream defence, for example, is not considered.

(2) The resistance of different defences to extreme loading is independent *i.e.* the strength of each defence is assessed independently and does not depend upon the strength of its neighbours. For a system of two defences, d_1 and d_2, that are both subjected to load l the probability of both defences failing is then:

$$P(D_1 \cap D_2|l) = P(D_1|l).P(D_2|l). \tag{4.6}$$

(3) The resistance within a given defence section is fully dependent *i.e.* the whole section responds to loads in the same way.

For very long defences the third assumption becomes difficult to sustain. Whilst the parameters describing defence resistance, for example crest height or geotechnical properties, will show strong dependency nearby, CUR and TAW (1990) suggest that over a distance greater than about 500m these parameters are more or less independent. Therefore for the purposes of the defence systems analysis, defences over 600m in length are split into sections 300-500m long.

Having accepted the assumptions outlined above, the probability of a typical failure scenario $D_1 \cap .. \cap D_r \cap \overline{D_{r+1}} \cap ... \cap \overline{D_n}$, is calculated as follows:

$$P\left(D_1 \cap ... \cap D_r \cap \overline{D_{r+1}} \cap ... \cap \overline{D_n}\right) =$$

$$\sum_{j=2}^{q-1}\left[P\left(L \geq \frac{l_j + l_{j+1}}{2}\right) - P\left(L \geq \frac{l_j + l_{j-1}}{2}\right)\right] P(D_1 \mid l_j)...P(D_r \mid l_j).P(\overline{D_{r+1}} \mid l_j)...P(\overline{D_n} \mid l_j) \qquad (4.7)$$

To understand the impact of defence failure it is also important to establish the mode of failure, be it breach or overtopping. The impacts of a defence being overtopped or breached can be quite different and so need to be considered separately in the flood risk calculation. Failure of defence d_i by overtopping is denoted as D_{iOT} and failure by breaching as D_{iB}. Non-failures of defence d_i are labelled $\overline{D_{iOT}}$ and $\overline{D_{iB}}$ respectively. Breaching and overtopping are not fully independent failure mechanisms; overtopping can often be the initiating mechanism for a defence breaching (Thomas and Hall, 1992). A defence can therefore be in one of four combinations of failure mechanisms:

(1) $\overline{D_{iB}} \cap \overline{D_{iOT}}$ represents the state of non-failure of the defence.

(2) $\overline{D_{iB}} \cap D_{iOT}$ represents failure by overtopping (as the defence has not breached as well).

(3) $D_{iB} \cap \overline{D_{iOT}}$ represents failure by breaching without overtopping occurring.

(4) $D_{iB} \cap D_{iOT}$ represents failure by breaching with overtopping.

The probability of failure by overtopping given a particular load l is labelled $P(D_{iOT}|l)$ and the probabilities of breaching, with or without overtopping, again given load l are labelled $P(D_{iB} \mid D_{iOT}, l)$ and $P\left(D_{iB} \mid \overline{D_{iOT}}, l\right)$ respectively. The probability of non-failure is therefore

$$P(\overline{D_{iOT}} \mid l).P(\overline{D_{iB}} \mid \overline{D_{iOT}}, l) \qquad (4.8)$$

Since $P\left(\overline{D_{iOT}} \mid l\right) = 1 - P\left(D_{iOT} \mid l\right)$ and $P(\overline{D_{iB}} \mid \overline{D_{iOT}}, l) = 1 - P(D_{iB} \mid \overline{D_{iOT}}, l)$ the probability of non-failure can be rewritten as

$$P(\overline{D_{iOT}}, \overline{D_{iB}} \mid l) = [1 - P(D_{iOT} \mid l)][1 - P(D_{iB} \mid D_{iOT}; l)] \qquad (4.9)$$

The probability of just overtopping, $P(D_{iOT}, \overline{D_{iB}} \mid l)$ occurring given load l is

$$P(D_{iOT}, \overline{D_{iB}} \mid l) = P(D_{iOT} \mid l) - P(D_{iB} \mid D_{iOT}, l).P(D_{iOT} \mid l) \qquad (4.10)$$

The probability of breaching given load l, $P\left(D_{iB} \mid l\right)$, is the sum of the probabilities of breaching occurring with overtopping and without overtopping

$$P\left(D_{iB} \mid l\right) = P\left(D_{iB} \mid D_{iOT}, l\right).P\left(D_{iOT} \mid l\right) + P\left(D_{iB} \mid \overline{D_{iOT}}, l\right)P\left(\overline{D_{iOT}} \mid l\right) \qquad (4.11)$$

which can be rewritten as:

$$P(D_{iB} \mid l) = P(D_{iB} \mid D_{iOT}, l)P(D_{iOT} \mid l) + P(D_{iB} \mid \overline{D_{iOT}}, l)[1 - P(D_{iOT} \mid l)]. \tag{4.12}$$

To evaluate Equations 4.9, 4.10 and 4.12 requires three fragility curves: (i) overtopping, $P(D_{iOT} \mid x)$, (ii) breaching given overtopping, $P(D_{iB} \mid D_{iOT}, x)$ and (iii) breaching given no overtopping, $P(D_{iB} \mid \overline{D_{iOT}}, x)$. Considering one typical defence failure scenario $D_{1OT} \cap \ldots \cap D_{rB} \cap \overline{D_{r+1}} \cap \ldots \cap \overline{D_n}$, Equation 4.7 becomes:

$$P(D_{1OT,\bar{B}} \cap \ldots \cap D_{rB} \cap \overline{D_{r+1}} \cap \ldots \cap \overline{D_n}) =$$

$$\sum_{j=1}^{q}\left[P\left(L \geq \frac{l_j + l_{j+1}}{2}\right) - P\left(X \geq \frac{l_j + l_{j-1}}{2}\right)\right]P(D_{1OT,\bar{B}} \mid l_j)..P(D_{rB} \mid l_j).P(\overline{D_{r+1}} \mid l_j)...P(\overline{D_n} \mid l_j) \tag{4.13}$$

where $P(D_{1OT,\bar{B}} \mid l_j)$ is obtained from Equation 4.10 and $P(D_{rB} \mid l_j)$ is obtained from Equation 4.12.

For each defence there are three states that are of interest: overtopped, breached and not failed. For a flood damage zone protected by n defences there are therefore 3^n system states whose probabilities are to be estimated, this becomes computationally infeasible for defence systems of a realistic size. A significant reduction in the number of defences can be considered if only those defences that result in some flooding in a given flood damage zone are considered. If the average number of defences protecting each zone, $z_1, \ldots, z_m$, is a, where a is much less than n, the number of failure scenarios to be considered reduces from 3^n to the order of $m.3^a$. Further saving is achievable if it is recognised that many of these failure scenarios will be common to neighbouring zones.

It may still be necessary to reduce the numbers of calculations further and this can be achieved by neglecting combinations of defence failure involving large numbers of defences as these make a small contribution to the total probability of failure. The error due to this approximation can be calculated exactly provided the probability of non failure for the whole system $P(\overline{D_1} \cap \overline{D_2} \cap \ldots \cap \overline{D_n})$ is also calculated. Suppose that in a system with n defence sections, the probabilities of all scenarios with between zero and five failures have been calculated. There will be

$$r = \sum_{i=0}^{5} \frac{n!}{i!(n-i)!} \tag{4.14}$$

such scenarios, the probability of each of which is labelled $P_j, j = 1, \ldots, r$. The error E from neglecting higher order scenarios is given by

$$E = 1 - \sum_{j=1}^{r} P_j \tag{4.15}$$

An upper bound on flood risk can therefore be calculated by multiplying this risk with the maximum possible damage within the flood damage area.

4.3.3. Constructing fragility curves

In a detailed risk analysis, fragility curves for overtopping and breaching mechanisms would be constructed on a defence-specific basis considering dimensions, material properties and failure mechanisms. However, the information required to construct defence-specific fragility curves in not available on a national basis. Generic fragility curves have therefore been constructed based on defence classification and a condition assessment. A new defence classification hierarchy is introduced which is used to establish generic defence type fragility curves for overtopping and breaching. There is a large element of expert judgement involved in the construction of these curves. This has been supplemented by additional analysis wherever possible.

Defence classification

In England and Wales the Environment Agency (1996) classifies every flood defence based on the individual defence components (for example inward slope, crest and outward slope) and their composition (for example turf or concrete). This leads to a classification in which sub-divisions have little relevance to the proneness to failure, whilst important characteristics such as crest width and level can go unrecorded.

For the purpose of national-scale risk analysis a simple new classification has been developed, focussing on those salient characteristics of a defence cross-section that influence its resistance to extreme loads. An algorithm has been established that gives a direct mapping from the classification used by the Environment Agency to the new reliability-based classification used here.

The highest level in the classification shows seven defence types that exhibit significantly different behaviour (Figure 4.3). The next level within the hierarchy considers the degree of protection offered by the defence; a wider defence will provide more protection, as will a defence that is protected on its front slope, crest and rear slope (Environment Agency, 1996b). The final level of classification considers the properties of individual components. A complete summary of the defence classification is given in Appendix E.

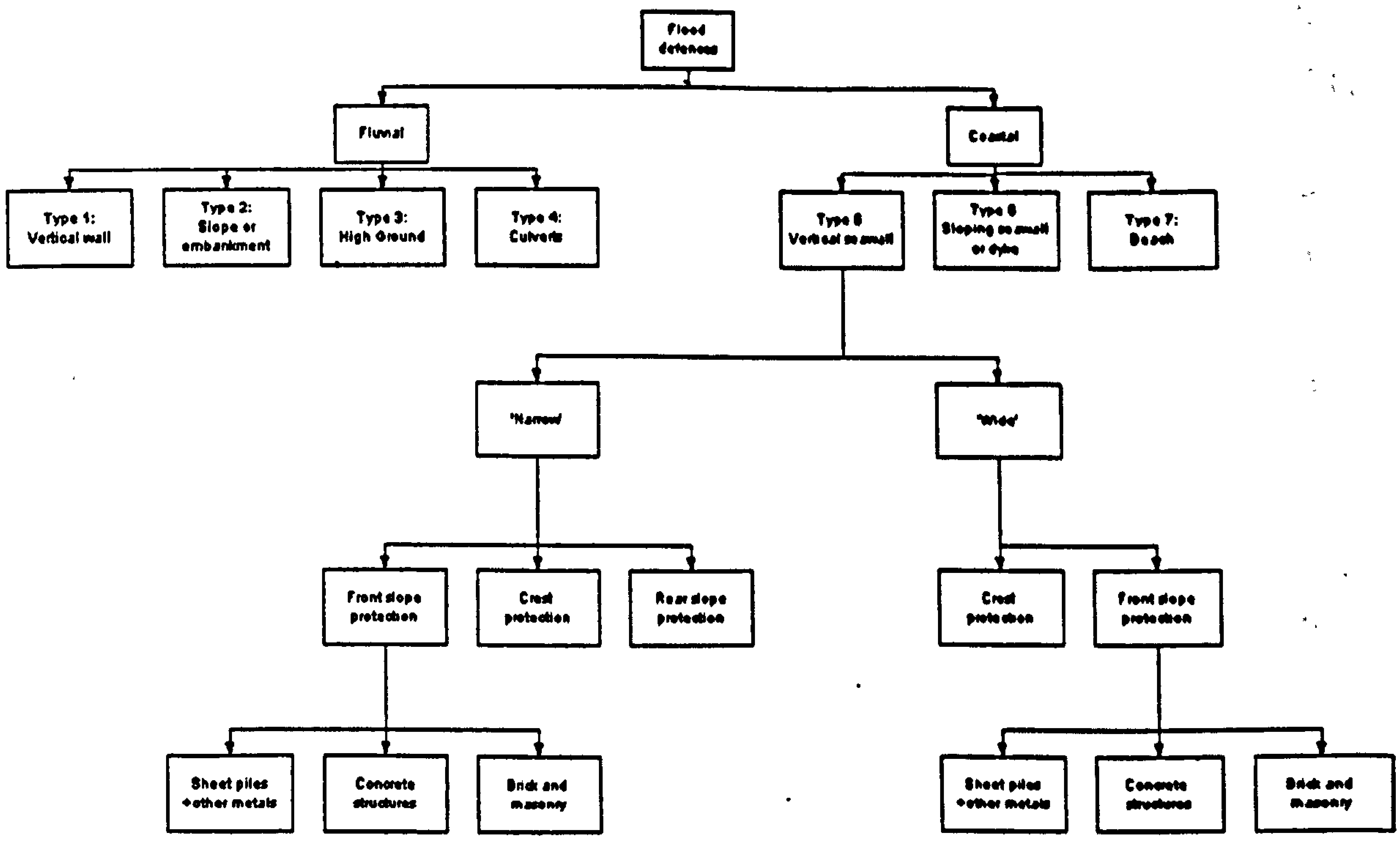


Figure 4.3 The seven main classes of flood defence. The classification for seawalls has been expanded as an example.

Overtopping mechanism

If the SOP were a perfect measure of the return period at which the defence would overflow or be overtopped then for all $x \geq 1$ the defence would have failed by overtopping and for all $x < 1$ it would be safe. However, flood defence designers typically include some allowance for freeboard, and there is also uncertainty in crest levels and extreme loads. This means that defences do not on the whole overtop as soon as $x = 1$. Freeboard allowance cannot be assumed to be nationally uniform, because of different local conventions and assumed rates of settlement. The fragility curve shown in Figure 4.4 has therefore been adopted. Uncertainty is reflected through the use of upper and lower bounds on the conditional failure probability for a given load, x.

The definition of SOP relates to the condition in which there is 'significant' overtopping. Interpretation of this definition of overtopping is subjective. In the case of fluvial defences, overtopping is dominated by overflow, as opposed to wave action (CIRIA, 1987), which is defined as the condition when the river stage exceeds the defence crest level. Evidence from floods of known severity overtopping defences of known SOP, primarily from records of the autumn/winter floods of 2000 (Environment Agency, 2001), has been used to verify points on the fragility curve shown in Figure 4.4. Additional statistical analysis of the level of freeboard showed that defences frequently have a substantial amount of freeboard (HR Wallingford *et al.*, 2002), significantly raising the level of protection they offer against overtopping compared to their design event, explaining the low conditional probability of inundation shown in Figure 4.4.

For sea and tidal defences, overtopping is dominated by wave action. Overflow is likely to lead to the breaching of the defences (see Appendix D for a review of failure mechanisms). In random seas in any storm there is a sometimes small but certainly finite probability of overtopping. The fragility curve for overtopping of sea defence shown in Figure 4.4 reflects this variability through increased uncertainty (although this uncertainty diminishes with greater loads).

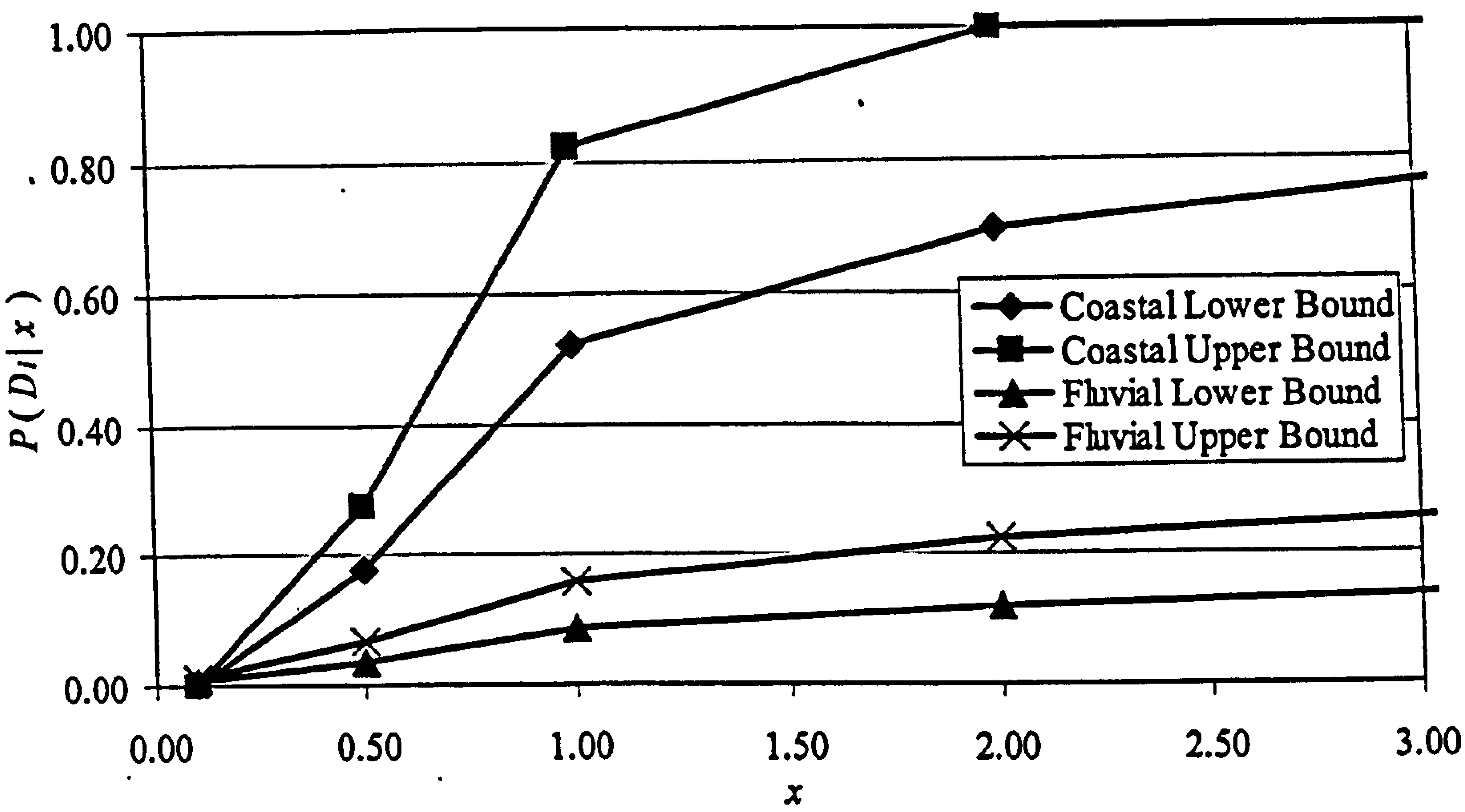


Figure 4.4 Fragility curve used in the high level analysis showing the conditional probability of overtopping for fluvial and coastal defences

Breaching mechanism

The probability of breaching in a storm of given severity is influenced by the type of defence and its condition. As suggested in Section 4.3.2, it is also strongly influenced by presence or absence of defence overtopping. Therefore a family of fragility curves have been developed for each defence classification, condition grade and overtopping/non-overtopping cases. An example of a family of these curves is shown in Figure 4.5. The fragility curves were are developed using a similar technique to that proposed by the USACE (1996) in which critical points on a curve are fixed by a combination of expert judgement and analysis, with a straight line between them.

The difficulty arises in the As described in Chapter 2, the only nationally available information on defence condition is a visual assessment that grades each defence from Grade 1 ("very good") to Grade 5 ("very poor"). The Environment Agency's Condition Assessment Manual (Glennie *et al.*, 1991) provides benchmark photographs of the main types of defence in all five conditions. Grade 5 nominally represents a defence in an effectively failed condition. However, the photographs (Figure 4.6 and Figure 4.7) in the Condition Assessment Manual indicate that some of these defences would afford some resistance against breaching, at least in loads where $x < 1$, so a fragility curve has been established based on assessment of this residual resistance. Unfortunately field evidence of defence failure is very scarce indeed. Verification of the fragility curves has therefore been based primarily on published values of the resistance of defence materials (CIRIA, 1987, CUR and TAW, 1990, CIRIA and CUR, 1991, Environment Agency, 1996b).

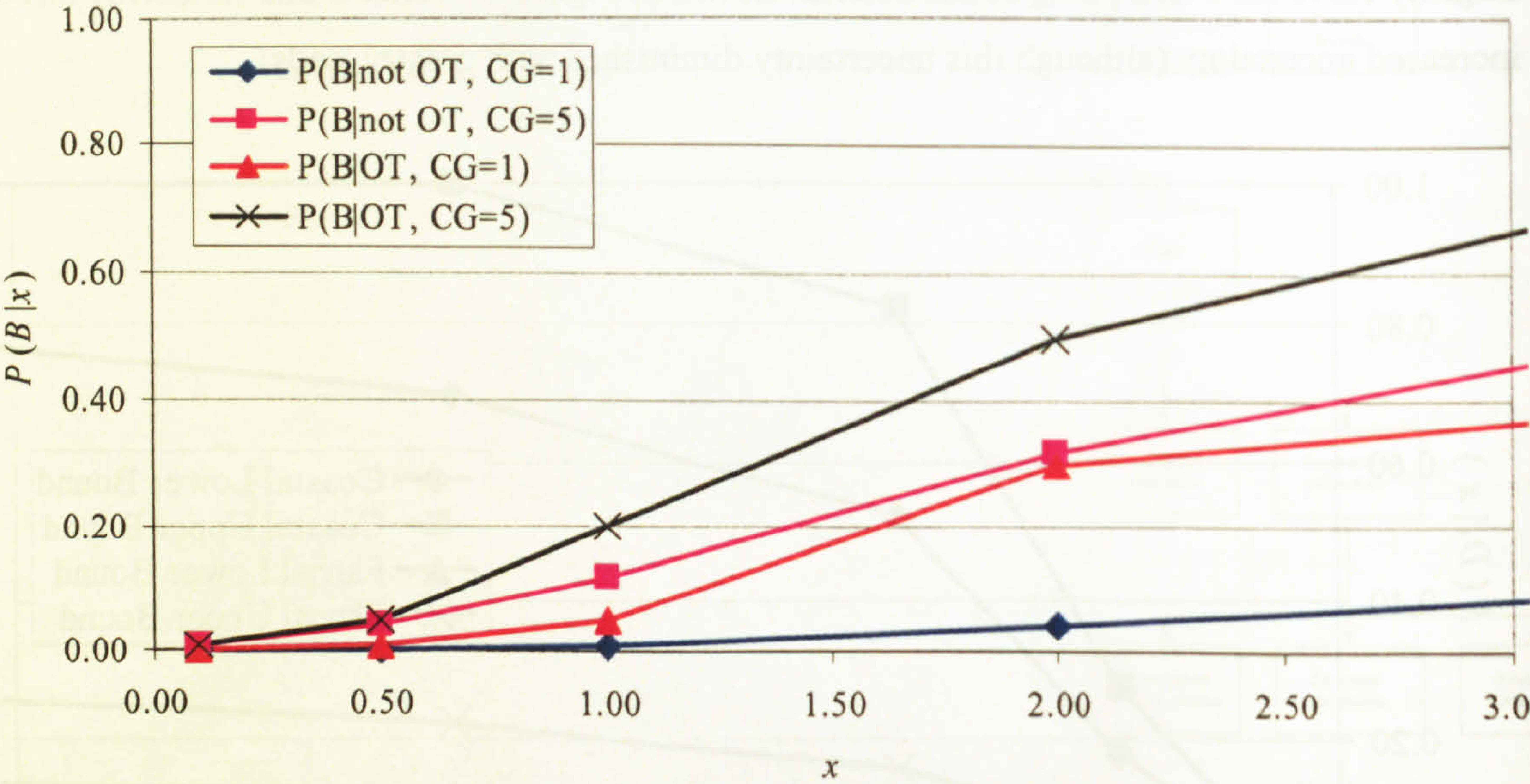


Figure 4.5 Fragility curve used in the high level analysis showing the conditional probability of breaching with and without overtopping for a seawall of condition grade 1 and 5

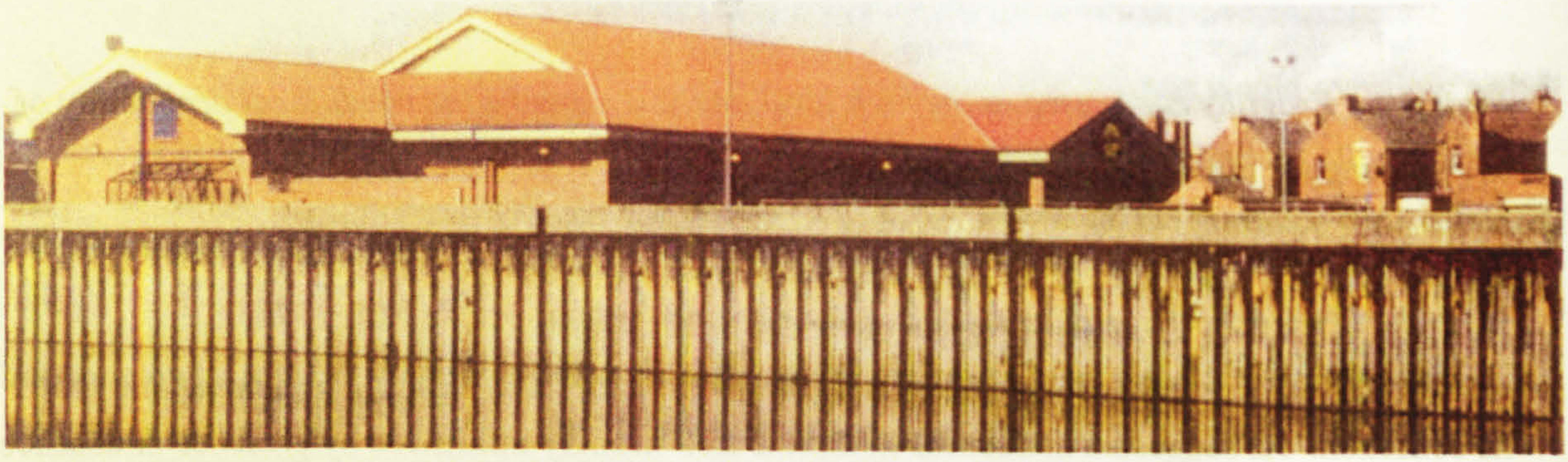


Figure 4.6 Sheet pile wall with a condition grade of "very good" (Glennie et al., 1991)

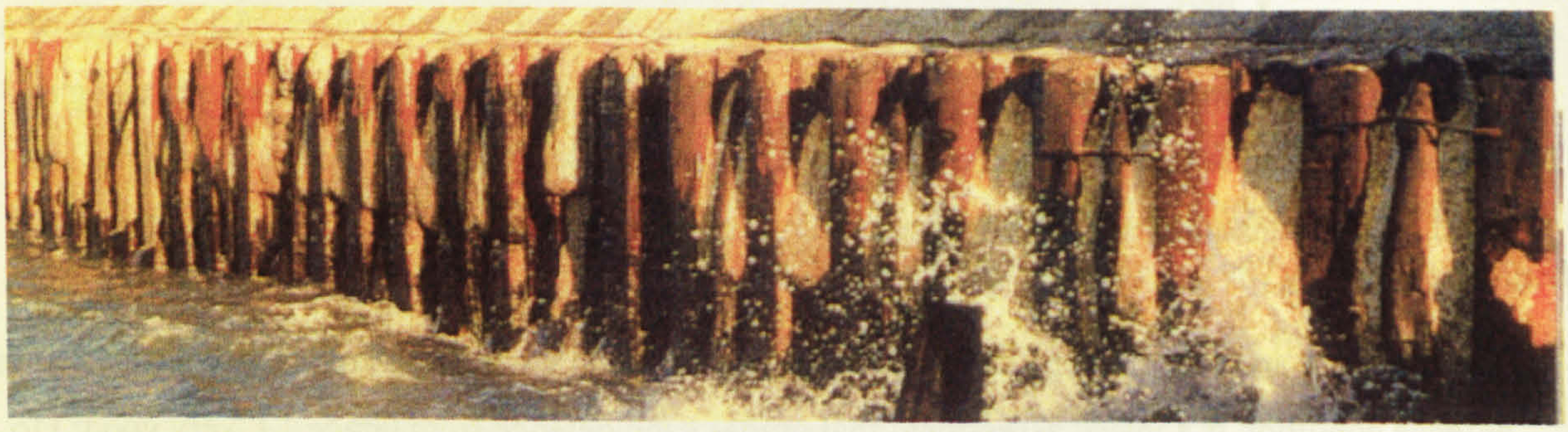


Figure 4.7 Sheet pile wall with a condition grade of "very poor". Note that the piles appear to offer plenty of residual strength despite being in the worst possible condition grade category (Glennie et al., 1991)

The bounds on the fragility curves have predominantly been elicited from expert judgement as the credible limits between which they expect the actual values to lie. As described earlier, limited evidence has been used to verify these bounds. No calibration of expert judgement using methods such as those proposed by Cooke (1991) has been applied, however, in order to address bias and over-confidence the judgement of several experts has been pooled. Fragility curves for each defence classification type are shown in Appendix E.

4.3.4. Inundation modelling

For each failure scenario an approximate flood outline is generated using parametric routines established by external collaborators. These routines estimate discharge through or over the defence. In the absence of topographic data and water levels, this is based on the valley type and statistical data of real and simulated flood events. An inundation outline and the corresponding depths in each damage zone are estimated for each defence failure scenario. These are used to construct probability-depth curves for each damage zone in the floodplain. The inundation methodology is described fully in Appendix F.

4.3.5. Evaluating risks

Damage zones within the floodplain were established based on a 1km square grid. The probability distribution of flood depth was calculated at the centroid of each damage zone and assumed to apply to the whole of the damage zone.

Economic damage

The numbers of domestic and commercial properties in each damage zone were extracted from nationally available databases. For a given damage zone the average annual damage, *AAD*, is given by:

$$R = \int_0^{y_{max}} p(y)D(y)dy \tag{4.16}$$

where y_{max} is the greatest flood depth from all failure scenarios, $p(y)$ is the probability density function for flood depth and $D(y)$ is the damage at depth y. The total expected annual damage for a catchment or nationally is obtained by summing the average annual damages for all damage zones.

Social impacts

The population at risk was estimated from the number of inhabitants within a damage zone using 2001 census data. The Social Flood Vulnerability Indices (SFVI) (Tapsell *et al.*, 2002) were used to identify communities vulnerable to the impacts of flooding. Social vulnerability is ranked from "very low" to "very high" and is based on a weighting of the number of lone parents, the population over 75 years old, the long term sick, non-homeowners, unemployed, non-car owners and overcrowding, obtained from census returns. The risk of social impact is obtained as a product of probability of flooding times the SFVI, providing a comparative measure for use in policy analysis.

Uncertainty in the risk assessment

Sections 4.3.2 to 4.3.4 involve applying methods that are prone to large amounts of uncertainty; both because of the approximations that have been made and the great variability in the quality of data available on a national scale.

A simple, yet explicit method of representing uncertainty at all the stages of the risk assessment has been applied. The uncertainty is represented by identifying an upper and lower bound of each of the most uncertain quantities. For example, the uncertainty in the fragility curves (Figure 4.4) is addressed by use of an upper and lower bound on the fragility curves, representing both the uncertainties in defence response and extreme loadings. The final outputs for flood risk are represented as upper and lower bounds. The uncertainties accounted for by these bounds are the uncertainty in loadings, defence response, inundation volumes and property damage. No assumption about the distribution of risk between these bounds can be made, though in the absence of more information, the best estimate will be the average of the two values. The wide bounds on risk estimates, in particular at a local resolution provide a cogent motive for a more thorough risk assessment to support decisions made at this level.

4.4. EXAMPLE IMPLEMENTATION: THE RIVER PARRETT AND BRIDGWATER BAY

Prior to national application (HR Wallingford *et al.*, 2003), the flood risk assessment methodology was first tested by the author on the Parrett catchment in the South West of England, a system of sufficient complexity to evaluate the robustness of the methodology, before proceeding to the full national assessment. The lower reaches of the Parrett include a network of drainage channels, whereas the upper reaches are quite steep. The fluvial defences adjoin short sections of sea defence in Bridgwater Bay. Establishing the defence system involved merging geographically indexed data on flood defences from the NFCDD (the Environment Agency database) with data on the centreline of all watercourses, held by CEH Wallingford. A continuous defence line on both banks of every watercourse and along all coastlines fronting the floodplain was generated. Significant lengths of river were not reported in the database as having a defence, in which case it was assumed that there was no raised defence. A non-raised defence can not fail by breaching and will provide resistance against overtopping equal at least to its neighbouring defences.

The two main outputs for the analysis, which were geographically indexed in a Geographical Information System (GIS), were the risk in each damage zone and the average contribution to this risk from each defence in the flood defence system. Figure 4.8 shows the output from the economic risk assessment. The damage zones have been shaded, with a darker shade representing a higher economic risk. The total expected annual damage for the Parrett catchment was calculated as £1.4-£2.1million. This compares with the only previous analysis (Halcrow *et al.*, 2001) which estimated a total risk of £2.7 million. In the close-up of the Parrett estuary shown in Figure 4.9 the defence line has been coloured to represent the contributions to risk from each defence. The darker shades represent a higher risk contribution (ranging from <£10 expected annual damage for some impact zones dominated by poor quality agricultural land to >£1,000,000 expected annual damage for high density urban areas). Defence failure probabilities (established using the fragility curves) can also be viewed in a similar manner.

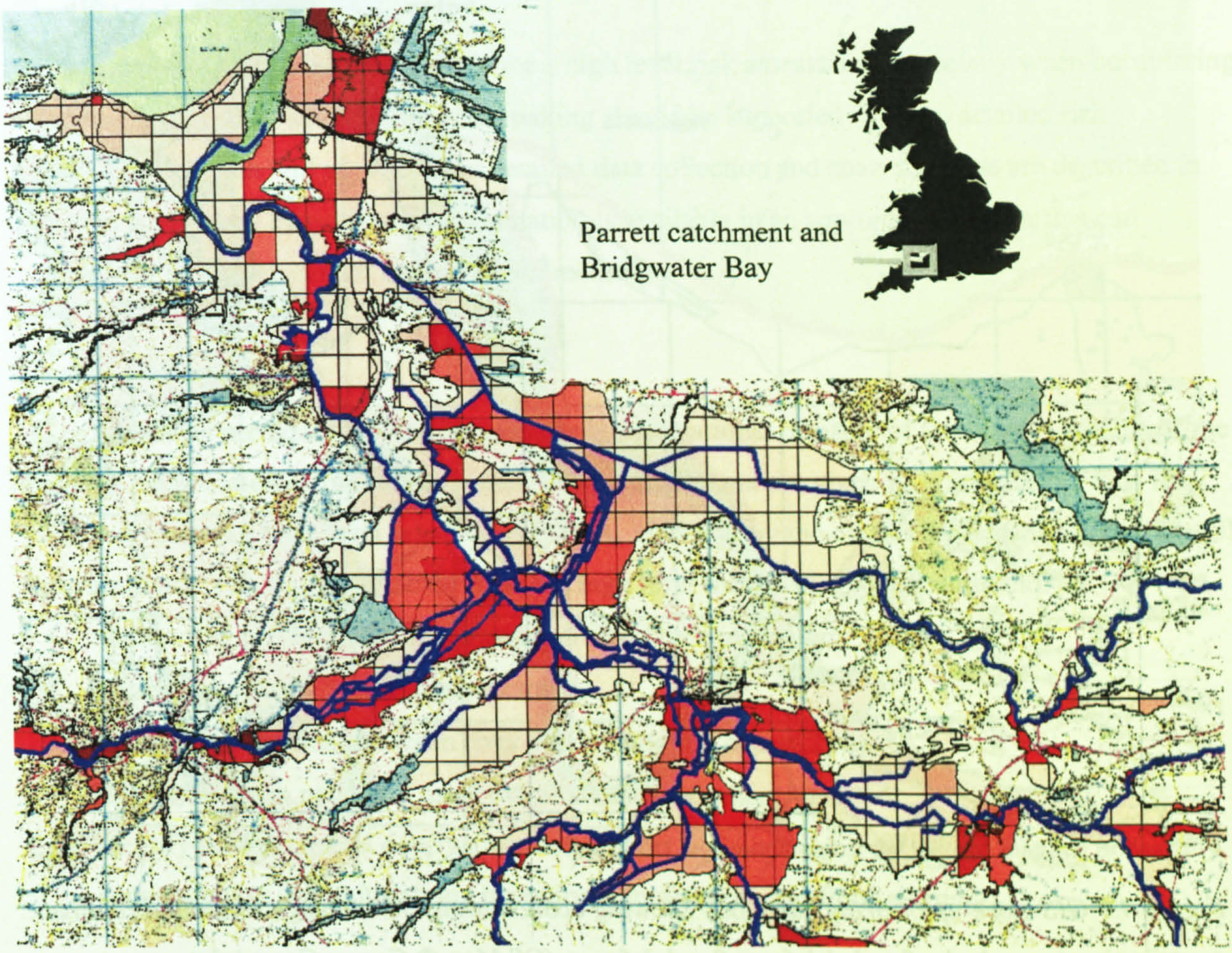


Figure 4.8 GIS view of flood risk for the Parrett catchment where darker shades represent greater flood risk (OS Map © Crown Copyright)

Other queries that can be made on the output data include:

- risk contribution from defences sorted by condition grade;
- risk contribution from defences sorted by SOP;
- risk contribution from floods of varying severity;
- number of people at risk from flooding and a measure of their social vulnerability;
- number of houses at risk of flooding to a given depth with a given probability. For example, in the Parrett catchment, between 2335 and 2704 residential properties (out of a possible 15668) and from 492 to 601 non residential properties (out of a possible 1605) are expected to be flooded to a depth of 0.2m or greater in a 1:100 year flood.
- number of properties with a high social vulnerability at risk of flooding to a given depth with a given probability. For example, in the Parrett catchment, between 229 and 458 people of

maximum vulnerability (out of a possible 1374 with the maximum vulnerability and 17273 total properties) are expected to be flooded to a depth of 0.2m or greater in a 1:100 year flood.

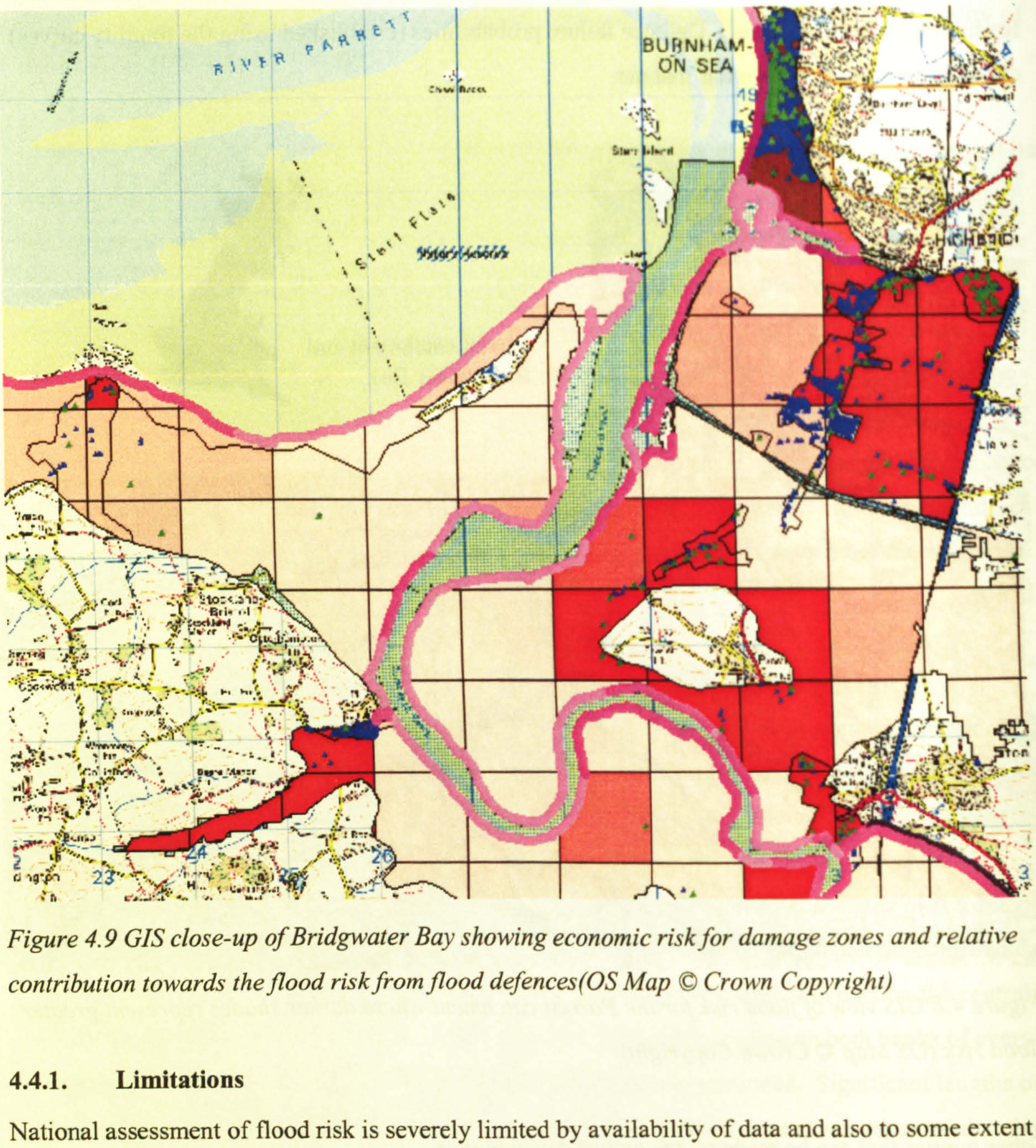


Figure 4.9 GIS close-up of Bridgwater Bay showing economic risk for damage zones and relative contribution towards the flood risk from flood defences(OS Map © Crown Copyright)

4.4.1. Limitations

National assessment of flood risk is severely limited by availability of data and also to some extent by computational constraints. The proposed high level risk assessment method has been designed to give an unbiased aggregate measure of risk on a national basis and cannot be expected to be consistently accurate for every locality. The key limitations are as follows:

- The frequency of extreme fluvial flows or marine storms has not been assessed directly. The factor x times SOP has been used as a proxy for load.
- Only linear defences are considered in the systems analysis.
- Probabilistic analysis of defence resistance using fragility curves is based on a simple defence classification and generic fragility curves that do not take explicit account of defence geometry and other key parameters that determine defence resistance.

- The current quality of information relating to defence location, condition and SOP within the Environment Agency database is highly variable.
- The flood spreading routine is based on volumetric concepts but does not include any hydrodynamic modelling and is based on a simple characterisation of floodplain morphology and approximate flood outlines in the IFM.
- Flood depths are based on statistical analysis of real and simulated data and do not take account of local topography.

These approximations are appropriate for a high level risk assessment, especially when considering the sparse data. Site specific decision-making should be supported by more detailed risk assessments which will require more detailed data collection and analysis, these are described in section 4.5. When more detailed information is available in an appropriate format, this can contribute to verification of the high level method.

4.4.2. Applications

The high level methodology can be used to obtain a national measure of flood risk. Results of the national application of the method described in this chapter are given in Appendix G. This is in line with meeting high level target 5b (DEFRA, 1999) set by the overseeing organisation of the UK government. The results obtained from the high level methodology can be used to:

- monitor the performance of the national flood defence system,
- justify investment from the government,
- prioritise investment between river catchments and coastal cells,
- identify areas in need of a more detailed risk assessment, and,
- identify areas of social deprivation and support decisions using indicators that are not purely economic (for use in justifying investment in defences and flood warning).

As well as providing an assessment of current risk, the methodology can be used to test future flooding scenarios and policy options, provided these can be resolved using the parameters of the methodology. For example, to illustrate the potential impact of climate change, the SOP of all defences was reduced by 20% (this reflects to an increase in water levels as a result of climate change). Therefore a defence that provided protection to a 1:200 year SOP effectively reduced to a 1:160 year SOP. In this climate change scenario the total economic risk increased to an expected annual damage of £1.8-£2.7million (Figure 4.10).

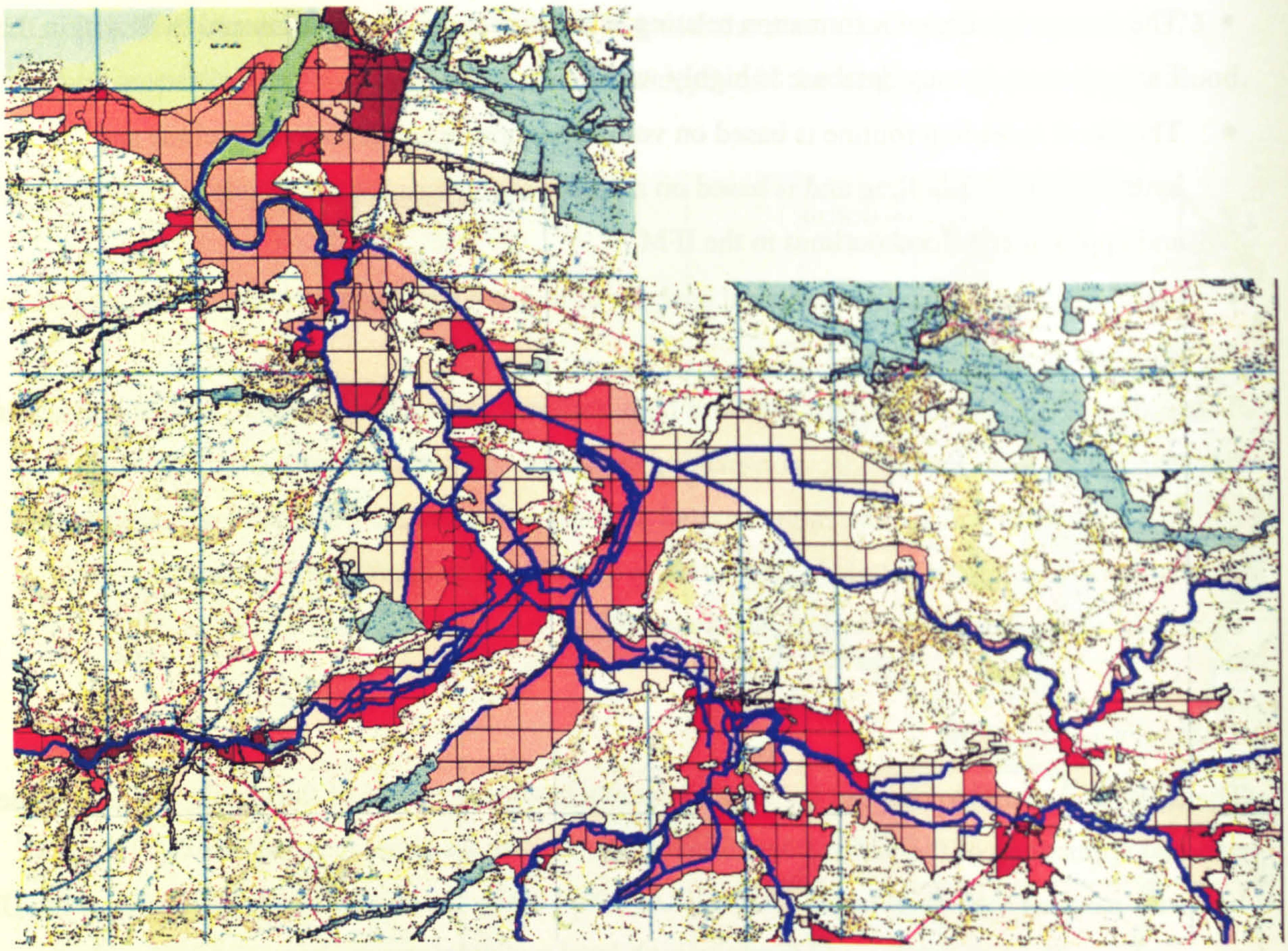


Figure 4.10 GIS representation of flood risk in the Parrett catchment after a climate change simulation had increased loads by 20% (OS Map © Crown Copyright)

The influence of repairing defences in poor condition can be assessed by altering the condition grade of defences. All defences with condition grade below 2 ("good") were raised to grade 2, reducing the total economic risk decreased to an expected annual damage of £1.3-£2.0million (Figure 4.11). This decrease is not particularly dramatic because most of the defences on the Parrett are already condition grade 2 or 3. The decrease in annual average risk can be weighed against the estimated cost of repairing the defences.

This clearly allows users to explore the influence of general implications of investment decisions. The approximate nature of the analysis means that it is not an appropriate methodology for use in detailed economic justification of schemes or strategy plans. However, a preliminary investigation may be performed using an approximate method to discard certain options at an early stage of the analysis.

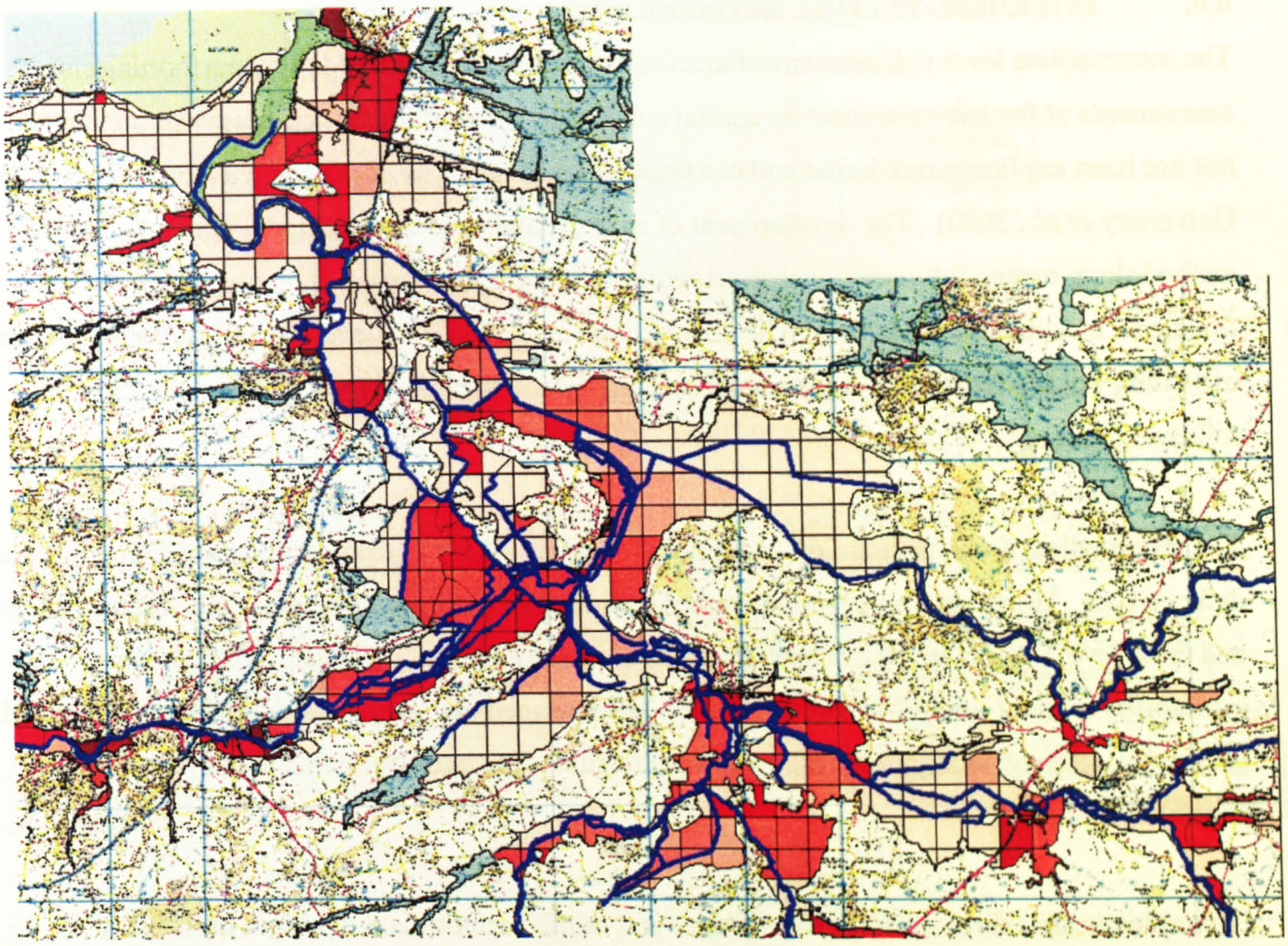


Figure 4.11 GIS representation of the Parrett catchment after maintenance had increased the condition grade of all defences to "good" (OS Map © Crown Copyright)

4.5. MORE RIGOROUS RISK ASSESSMENT

Section 4.4.1 identified a number of limitations with the high level methodology. These are addressed in more detailed analysis through:

- Statistical analysis of hydrology, and joint probability loading conditions for sea defences, including spatial dependency in both cases.
- Inclusion of additional components of the defence system, such as upstream storage reservoirs, pumps and relief channels to support more strategic decisions.
- Quantified probabilistic analysis of defence failure making use of site-specific measurements.
- Analysis of the dependency between defence strength parameters within defence sections and between neighbouring sections.
- Hydrodynamic modelling for flood depth and extent using high resolution topographic information.
- More detailed analysis of tangible and intangible impacts of flooding, including disruption to transportation systems. Analysis of the influence of non-structural flood mitigation measures such as flood warning.

The extent to which these are implemented in the intermediate and detailed level risk assessment is described in the following sections.

4.6. INTERMEDIATE LEVEL METHODOLOGY

The intermediate level risk assessment aims to support SMPs and CFMPs by performing risk assessments at the sub-catchment or coastal sub-cell level. Previous risk assessment on this scale has not been explicitly risk-based and consisted of varying levels of technical analysis (Newcastle University *et al.*, 2000). The development of a robust and accessible regional scale risk assessment methodology means future regional management plans can be supported by quantified estimates of flood risk in a consistent manner. The intermediate level has been developed fully, but at the time of writing not implemented. It is summarised briefly and then described in more detail in the following sections.

The intermediate level methodology continues to use data such as the defence condition grade and defence type. However, this is supplemented by additional information, including, defence width and crest level (which are also stored in the Environment Agency database, although their availability is more limited) where available. The intermediate level is not intended to be applied nationally, although computing resources and adequate data permitting this may eventually be possible..

Fluvial loads are estimated from river gauges and rainfall-runoff data to obtain probability distributions of inflow. Joint-probabilities of wave height and water levels are used to describe coastal loadings.

A hydrodynamic model incorporating a DEM is used to estimate inundation extents and depths in the floodplain. Damages are calculated using spatially indexed property and flood depth information.

Defence strength is described using fragility curves. These are constructed by identifying dominant failure modes and correlating the proneness of failure to known parameters such as geometry.

The possible number of combinations of defence failure in a system consisting of n defences equates to 2^n. To calculate a precise value of flood risk (or precise bounds on risk after accounting for uncertainties) it is necessary to perform a hydrodynamic simulation of all these scenarios. This is not possible so an algorithm has been established that identifies upper and lower bounds on flood risk. These bounds are then converged in the most efficient manner possible.

An overview of the Intermediate Level risk assessment methodology is shown in Figure 4.12. The methodology is described in more detail in the following sections.

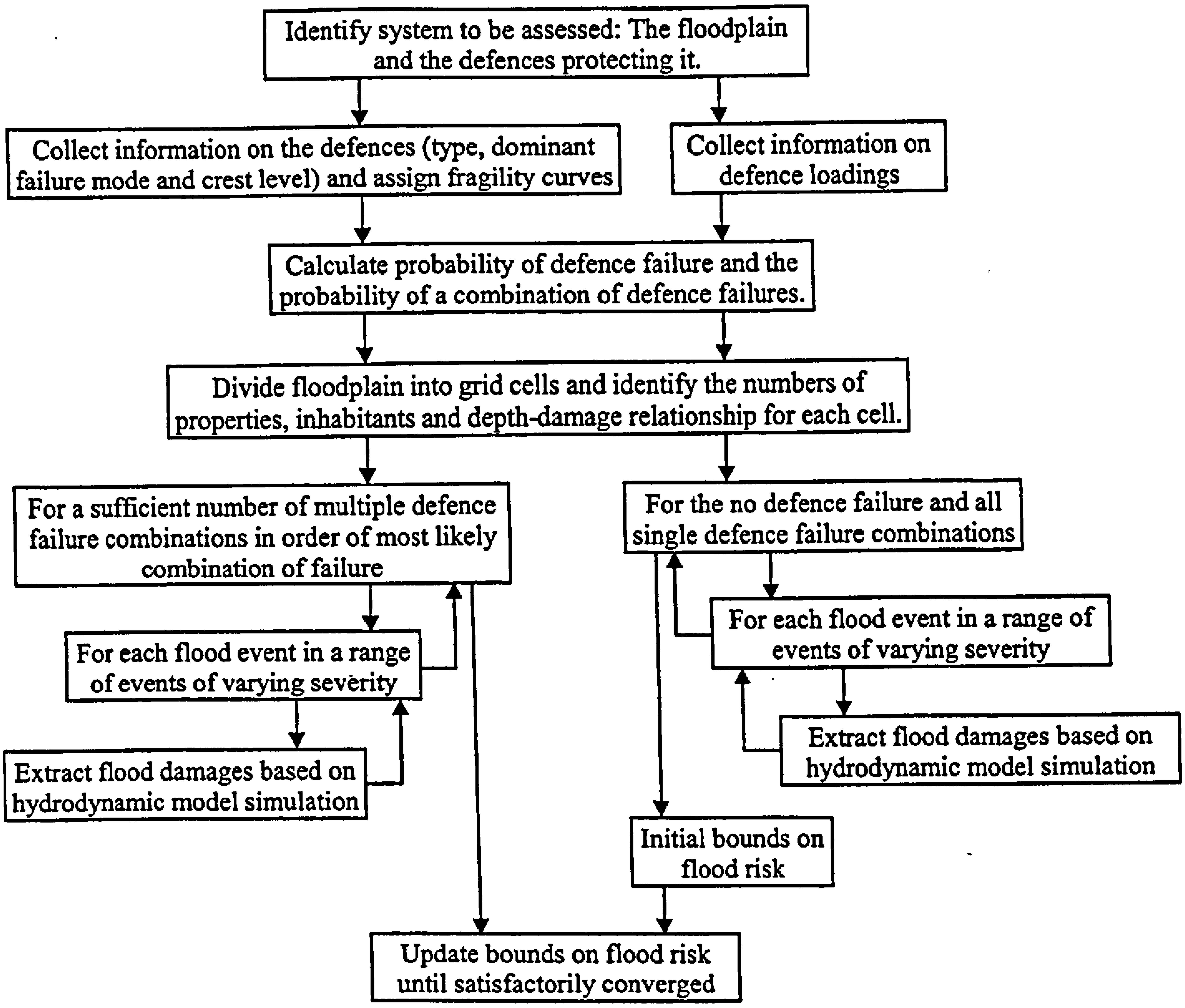


Figure 4.12 Overview of the intermediate flood risk assessment methodology

4.6.1. Load estimation

To estimate fluvial loads for the intermediate level, statistical analysis of recorded flow rates is used to compliment the FEH rainfall-runoff method (CEHW, 1999). These are used to obtain the probability, P, of a given inflow, Q.

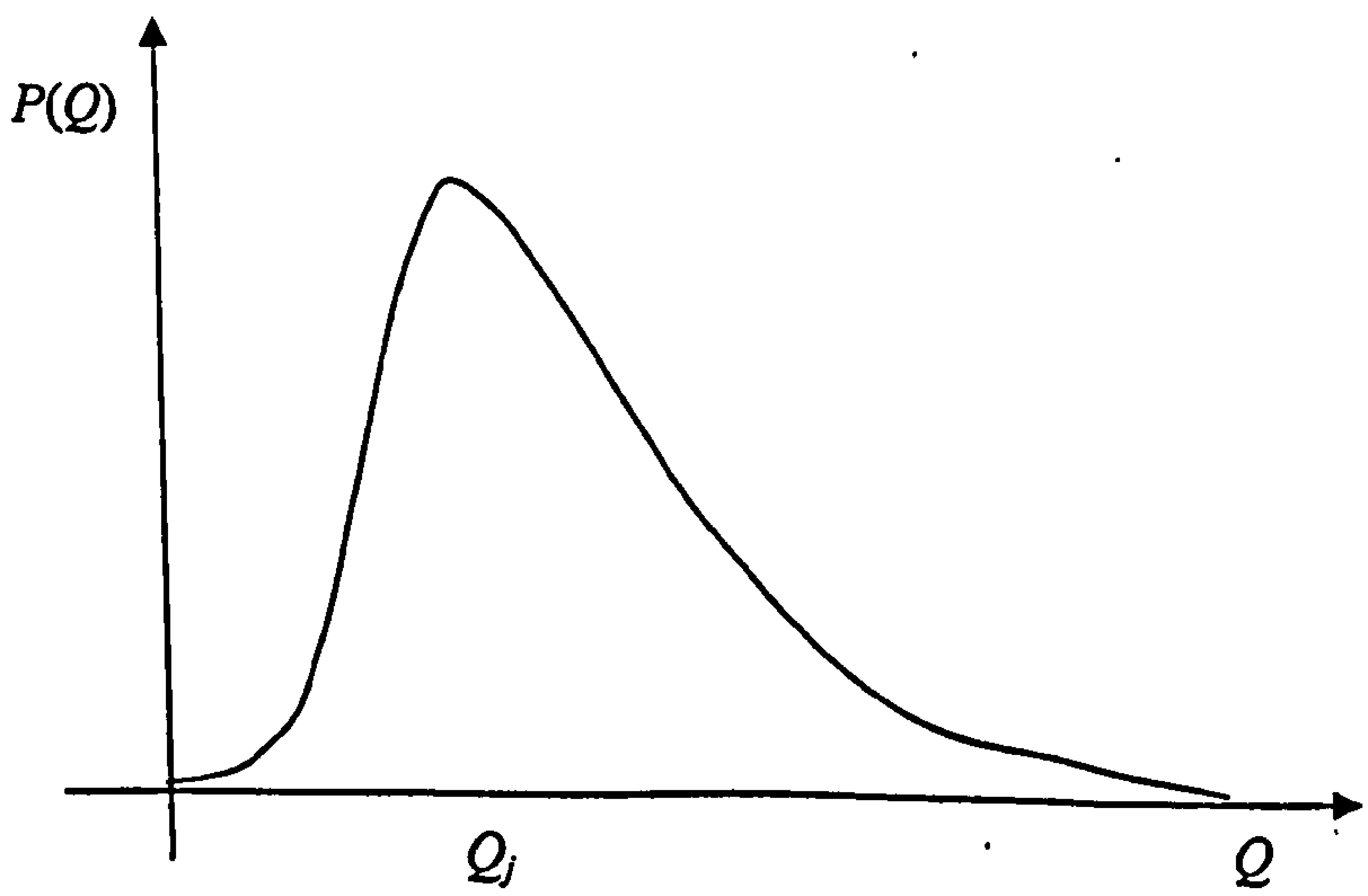


Figure 4.13 Example of a probability density function for inflow

For coastal flooding, joint probabilities of wave heights and water levels are needed. These are derived using methods described by HR Wallingford and Lancaster University (2000). Wave data is obtained from wave gauges and wind hindcasting. Extreme water levels are taken from POL 112 (Dixon and Tawn, 1997) and complimented by local measurements where available. Overtopping discharge rates for coastal defences are estimated using methods described in the overtopping manual (HR Wallingford, 1999).

4.6.2. Hydrodynamic modelling

Hydraulic modelling can be significantly improved from the high level methodology by using hydrological loading information, defence crest levels and a Digital Elevation Map (DEM) of the floodplain. Loads on defences are no longer considered to be identical for the entire system. However, the conservative assumption that defences fail simultaneously in multiple failure scenarios is made to enable the risk assessment to be performed at a large scale. The analysis is therefore event based rather than time-dependent. For each failure scenario the inundation model provides spatially indexed data on flood depths and extent which can be used to estimate damages. Whilst the risk assessment methodology is not restricted to a particular inundation model, a desirable characteristic is that the model can be set up to perform a large number of simulations rapidly and with little external interference.

4.6.3. Describing defence performance

Fluvial defence overtopping

For fluvial systems, overtopping is modelled within the hydrodynamic model. Depending on the hydrodynamic model being used, flow over a defence may be modelled using a weir equation (Chadwick and Morfett, 1993). For free flow (modular) when $h_u / h_d > m$:

$$Q = C_d \, b \, h_u^{1.5} \tag{4.17}$$

and for drowned flow when $h_u / h_d =< m$:

$$Q = \left(\frac{C_d \, b \, h_u \sqrt{(h_u - h_d)}}{\sqrt{(1-m)}} \right) \tag{4.18}$$

where Q = flow (m3/s), C_d = discharge coefficient , b = breadth of the weir, h_u = upstream depth of water above the embankment crest, h_d = downstream depth of water above the embankment crest, and, m = modular limit. This approach may be more suited to the 1-D or quasi-2-D models described in Section 3.3. If a depth-averaged 2-D model or full 3-D model is being used, then flow over the defence can be modelled by solving the Navier-Stokes equations (Section 3.3).

Coastal defence overtopping

For coastal defences, information on defence roughness is required to model overtopping using Owen's equation (HR Wallingford, 1980) (Equation 4.19) and later adaptations (HR Wallingford, 1999). For sloping, impermeable seawalls the overtopping rate, Q, is given by:

$$Q = gH_s T_m a \exp\left[\frac{-b(h_c - h_w)}{T_m\sqrt{gH_s}}\right] \tag{4.19}$$

wher H_s is the significant wave height, T_m is the mean wave period, h_c is the crest level, h_w is the still water level and a and b are coefficients based on the slope and berm. For rough and armoured slopes, a roughness coefficient is required. This is based on the type of material and where this is unknown a value can be assigned based on the limited defence revetment material classification information used for high level risk assessment (*eg.* turf front face corresponds to a roughness coefficient of 0.9-1.0).

Fluvial defence breaching

As at the high level method, defence condition is described using fragility curves (Casciati and Faravelli, 1991 and Chapter 5). In order to maximise the use of additional information, fragility curves are generated by identifying the dominant initiating failure mechanism of a particular structure. This is achieved through an onsite inspection of each individual defence. For example, for a fluvial defence this will usually be overflow leading to breaching or piping. Simple algebraic expressions using a limited number of parameters are used to construct fragility curves. In the case of overflow of fluvial defences, fragility curves can be constructed by relating conditional failure probabilities with overflow rates. Bettes and Reeve (1995) established relationship between the overflow head and the expected damage for embankments. This relationship can be used to help define points on the fragility curve. For mechanisms that are less well studied, relationships may have to be established based solely on expert judgement. An example of a fragility curve constructed through for an embankment most likely to fail from damage cause by overflow is shown in Figure 4.14. As at the high level, uncertainty in the defence response is represented by an upper and lower bound on the fragility curves.

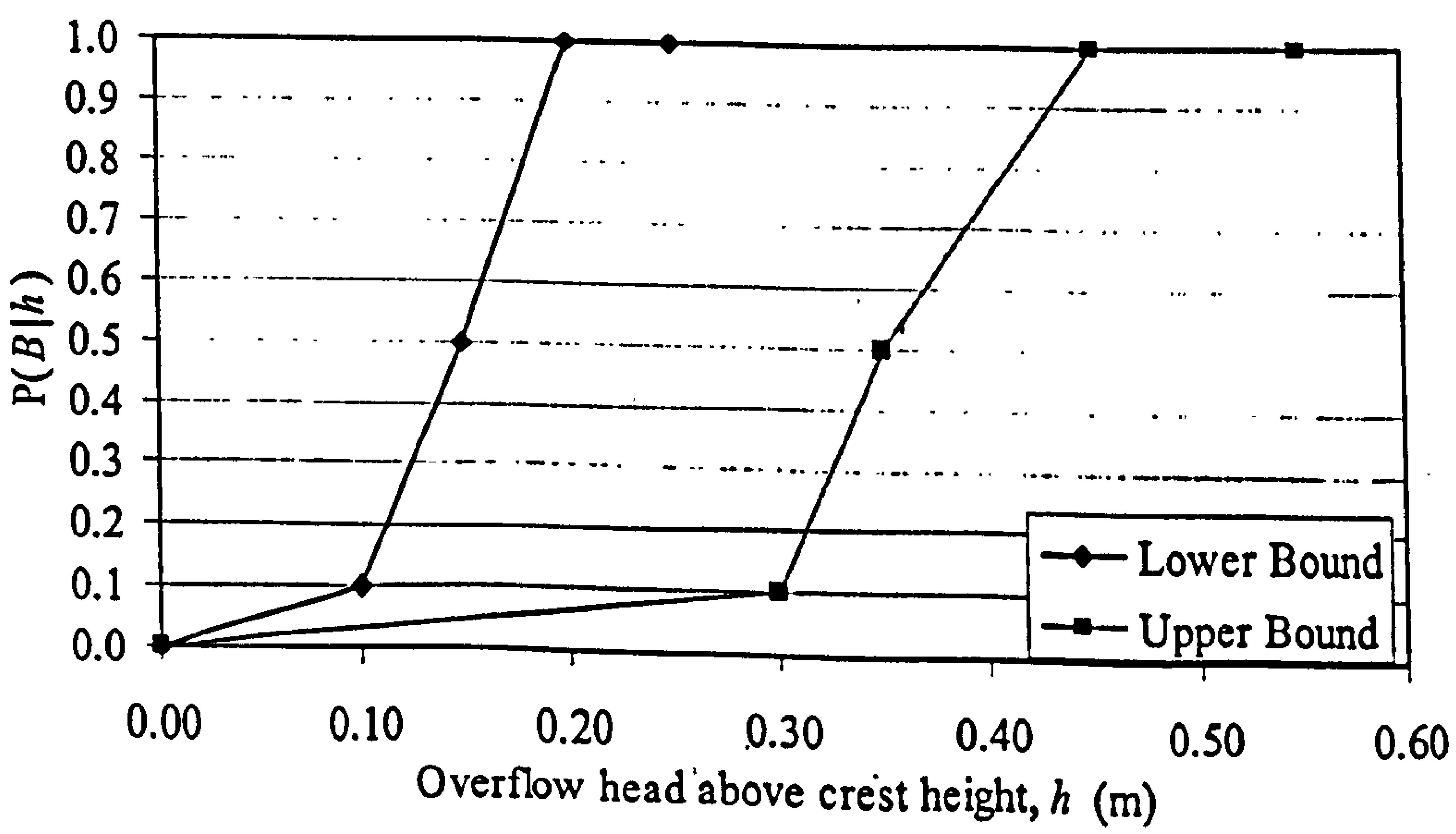


Figure 4.14 Fragility curve for an embankment with a dominant failure mode of damage caused by overflowing

Coastal defence breaching

Coastal defences are subjected to joint loadings of waves and water levels. For the intermediate level, as with fluvial defences, a dominant failure mode is identified. However, for mechanisms dependent on waves and water levels, a fragility surface is generated by plotting $P(B|WL,H_s)$ against the loading WL (water level) along one axis and H_s (wave height) along another.

Coastal morphology plays an important part in the proneness of a defence to breaching; a large and stable beach will offer additional protection. Intervention in coastal processes alters those downdrift. Assessment of changes in flood risk over the duration of a SMP requires the modelling of morphology to be incorporated into the risk assessment methodology. This enables intervention strategies, and their subsequent impact on morphology and flood risk to be tested. Methods available to model coastal morphology have been introduced in Chapter 3.

4.6.4. Defence systems failure analysis

The defence systems failure analysis is more rigorous than the high level methodology. Upstream storage (and other defences besides embankments such as sluice gates) are included alongside linear defences in the modelling of the system allowing a more complete range of strategic decisions to be supported. The methodology will be described in the context of fluvial systems. A few changes that are required for coastal systems are described later.

Defences can be in one of three states; overtopping, breaching or not failed. At the intermediate level overtopping volumes are given by the hydrodynamic model meaning that the number of possible combinations of defence failure (henceforth referred to as scenarios) that need to be analysed is now 2^n, for a system with n defences. The large number of failure scenarios means for any realistic system these scenarios can not all be modelled. Bounds on flood risk are therefore established. As more failure scenarios are modelled these bounds converge.

1. Establish 'worst case' water level loadings

The first step is to model the 'worst case' scenario of no defences failing – effectively establishing a baseline for system behaviour (*Scenario 0*). This establishes the worst case water level, WL_j, loading at each defence along the watercourse for the inflow, Q_j. Failure of any defence will serve only to potentially lower the water level at a particular defence for any given flood event. The flood extent, depths and damages accrued from any overtopping are also calculated.

2. Calculate failure probabilities for each scenario

Because of the potentially enormous number of scenarios it is not be possible to model the inundation of each failure scenario. It may not necessarily be possible to calculate the failure probability of each scenario. Therefore, the probability of each scenario up to the m^{th} order (*i.e. m* defences failing simultaneously) is calculated. For an n defence system the probability of a defence failure scenario, $P(S)$, needs to be calculated for r different scenarios, where r is defined as:

$$r = \sum_{i=1}^{m} \frac{n!}{i!(n-i)!} \tag{4.20}$$

The probability of a typical scenario (in this case involving the failure of defence d) is given by:

$$P(S_j) = P(\overline{D_1}) \times P(\overline{D_2}) \ldots \times P(D_d) \ldots \times P(\overline{D_{n-1}}) \times P(\overline{D_n}) \tag{4.21}$$

where $\overline{D_n}$ represents non-failure of defence n.

As described in Section 4.6.2 no account is made of the sequence of failure in multiple defence failure scenarios. This results in a conservative estimate of flood risk as any relief in water levels following failure of upstream defences is not considered in the estimation of the breach probability (though it is included in the hydrodynamic modelling of inundation extent). The length of time of loading is not considered either. Although a longer loading will increase the failure probability of a defence, this can be reflected in the fragility curve as a more serious flood event will usually result in a longer loading time on the defences. Both of these factors will be considered at the detailed level of the risk assessment methodology.

3. Estimate maximum possible damage

A maximum damage value is estimated by super-imposing the flood outlines and worst case flood depth in each floodplain cell from all single defence failure scenarios and the non-failure scenario. For each inflow estimate, j, the consequences, $Cmax_j$, from this extreme flood envelope are calculated. Note that this does not necessarily equate to the sum of all damage values for each scenario.

4. Calculate risk from single defence failure scenarios

The next stage involves simulating failure of each defence along the watercourse and using the flood outline and flood depths from the hydrodynamic model to establish the damages associated with this failure. The economic risk, R, for any given defence failure scenario is the multiple of its probability, $P(S)$, and consequences, C:

$$R = P(S) \times C \tag{4.22}$$

5. Calculate initial bounds on flood risk

A lower bound on the flood risk, R_L of a system with n defences can therefore be calculated by summing the risk for all first order scenarios:

$$R_L = \sum_{i=0}^{r} P(S_i) \times C_i \tag{4.23}$$

Because the probability of all the scenarios sums to unity an upper bound on the risk, R_U, can be calculated:

$$R_U = R_L + C_{max\,j} \times \left[1 - \sum_{i=0}^{r} P(S_i)\right] \tag{4.24}$$

This represents the sum of the risk associated with all first order failure scenarios and the conservative assumption that damage from all the remaining scenarios is the damage calculated from the super-imposed flood extent of the first order failure scenarios.

6. Rank remaining scenarios

The remaining failure scenarios (2nd order to m^{th} order) are ranked according to their probability of occurrence. Starting with the highest scenario probability the inundation model is run and the damage for each scenario is calculated. As each of these remaining scenarios is analysed, the bounds on flood risk will converge. If z represents the number of multiple defence failure scenarios analysed, the new bounds, R_{L+Z} and R_{U+Z} are now:

$$R_{L+Z} = \sum_{i=0}^{n+z} C_i \times P(S_i) \tag{4.25}$$

$$R_{U+Z} = R_{L+Z} + C_{max\,j} \times \left[1 - \sum_{i=0}^{n+z} P(S_i)\right] \tag{4.26}$$

The bounds will therefore converge by $C_{maxj} \times P(S_z)$ for each time an additional scenario is run. Enough scenarios should be calculated to ensure this convergence is satisfactory given the available resources and acceptable level of uncertainty.

The scenarios are ranked on probability as the term $C_{max\,j} \times \left[1 - \sum_{i=0}^{n+z} P(S_i)\right]$ is minimised by maximising the total probability analysed, $\sum_{i=0}^{n+z} P(S_i)$. This will act to reduce the upper bound in the most efficient manner possible thereby optimising the use of computational resources.

7. Calculate bounds of system risk

Performing this analysis for many values of Q will enable the upper and lower bounds on flood risk to be plotted against discharge Q (Figure 4.15).

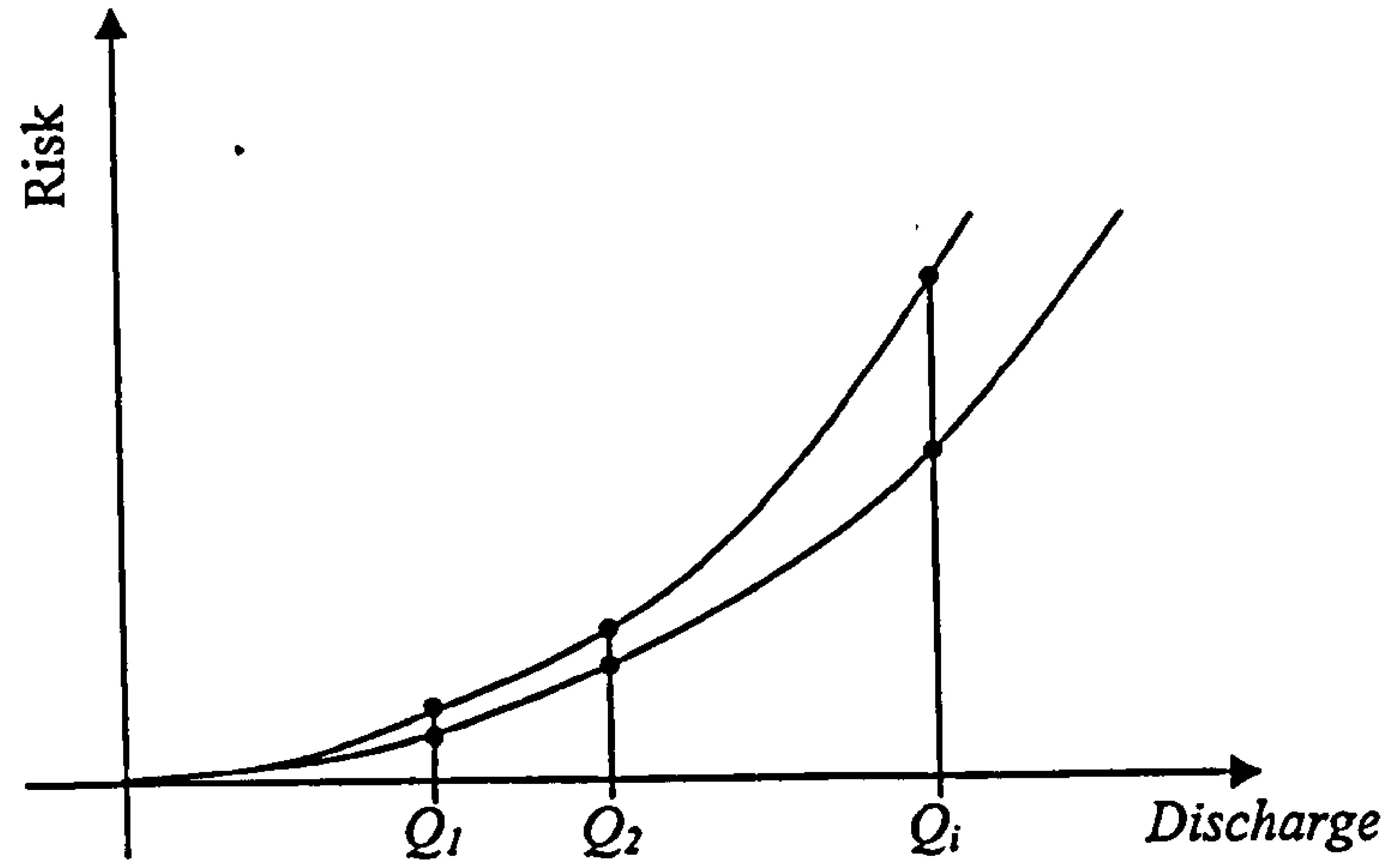


Figure 4.15 Plot of flood risk vs. river discharge Q

The plot of flood risk against the discharge, Q (Figure 4.15) is then integrated over the discharge distribution shown in Figure 4.13 to calculate an upper and lower bound for the flood risk of the system.

8. Coastal systems

The nature of coastal systems is different to that of fluvial systems. Both the directionality and the dual nature of the loading need to be considered. The risk assessment methodology for coastal systems takes the same general form, but with a small number of changes to reflect the differences between the two types of system.

Because coastal systems are subjected to both wave and water level loadings, the conditions most likely to result in defence failure are not necessarily the same as those likely to result in the maximum inundation. Therefore both extreme wave and water level events (as well as those in between) need to be considered.

The shape of the coastline often means that nearby defences protecting the same land are not necessarily subjected to the same loading conditions due to the directionality of the loading. An extreme example of this is shown in Figure 4.16. The methodology for accounting for the direction and type of loading is summarised as follows.

(1) Identify the system of n sea defences that protect a self-contained floodplain. Describe the resistance of each defence i ($i = 1,\ldots, n$) in the system in terms of a fragility function, conditional upon overtopping rate Q_i: $F(D_i|Q_i)$ where D_i denotes failure of defence i (Dawson and Hall, 2003*a*, 2003*b*).

(2) Construct a joint probability density function (j.p.d.f.) $f(H_s, WL)$ using simultaneous measurements of wave height H_s and water level WL at the site.

(3) Make a random sample of a large number (~10,000) points from the j.p.d.f. $f(H_s, WL)$. For each point in this sample calculate the overtopping rate, $Q_i(H_s, WL)$ at each defence i =

1,..., n in the system using the Overtopping Manual (HR Wallingford, 1999). Since parametric overtopping calculations are computationally inexpensive, no importance sampling is required at this stage.

(4) For each sample of (H_s, WL) estimate the conditional probability of system failure, $P(D_s|H_s, WL)$ assuming independence between defence sections:

$$P(D_s \mid H_s, WL) = \prod_{i=1}^{n}\left[1 - P(D_i \mid Q_i(H_s, WL))\right] \tag{4.27}$$

(5) Identify the point t on $H_s \times WL$ that maximises $P(D_s|H_s, WL).f(H_s \times WL)$. Make a fairly small sample of m points ($m \cong 100$) over a regular grid centred on t.

(6) For each point $j = 1,..., m$ calculate the conditional probability $P(D_k|H_s, WL)$, $k = 1,..., 2^n$ of all of the defence failure combinations (again, these calculations are computationally inexpensive). Select the r failure combinations that make a non-negligible contribution to the total conditional probability of system failure $P(D_s|H_s, WL)$ (generally $r \lll 2^n$).

(7) For each failure combination $k = 1,..., r$ run the inundation model using the loading conditions (H_s, WL), the overtopping rates the overtopping rates $Q_i(H_s, WL)$ and an empirical estimate of the breach size and dischage, for breaching failure modes (HR Wallingford, 2003).

(8) For each run $k = 1,..., r$ of the inundation model estimate the economic damage E_k using a database of house locations and standard depth-damage criteria.

(9) The conditional risk $R(H_s, WL)$ is given by

$$R(H_s, WL) = \sum_{k=1}^{r} P(D_k \mid H_s, WL).E_k \tag{4.28}$$

(10)Plot at each point $k = 1,..., r$ on $H_s \times WL$ the quantity $f(H_s, WL).R(H_s, WL)$. These points are then used to estimate the risk-based importance sampling distribution. Fit a joint p.d.f. $f_{imp}(H_s, WL)$ (normalising as necessary) to the values at $k = 1,..., r$.

(11)Sample as may points as are computationally feasible (typically 1000s) from $f_{imp}(H_s, WL)$. At each point repeat steps 6-8.

(12)The total flood risk R_{tot} is given by

$$R_{tot} = \iint R(H_s, WL) f(H_s, WL) dH_s dWL \tag{4.29}$$

which may be obtained by numerically integrating the results obtained in Step 11 with the j.p.d.f. $f(H_s, WL)$.

These steps can be repeated twice using upper and lower values of crest heights, damage values *etc.* to obtain bounds on flood risk.

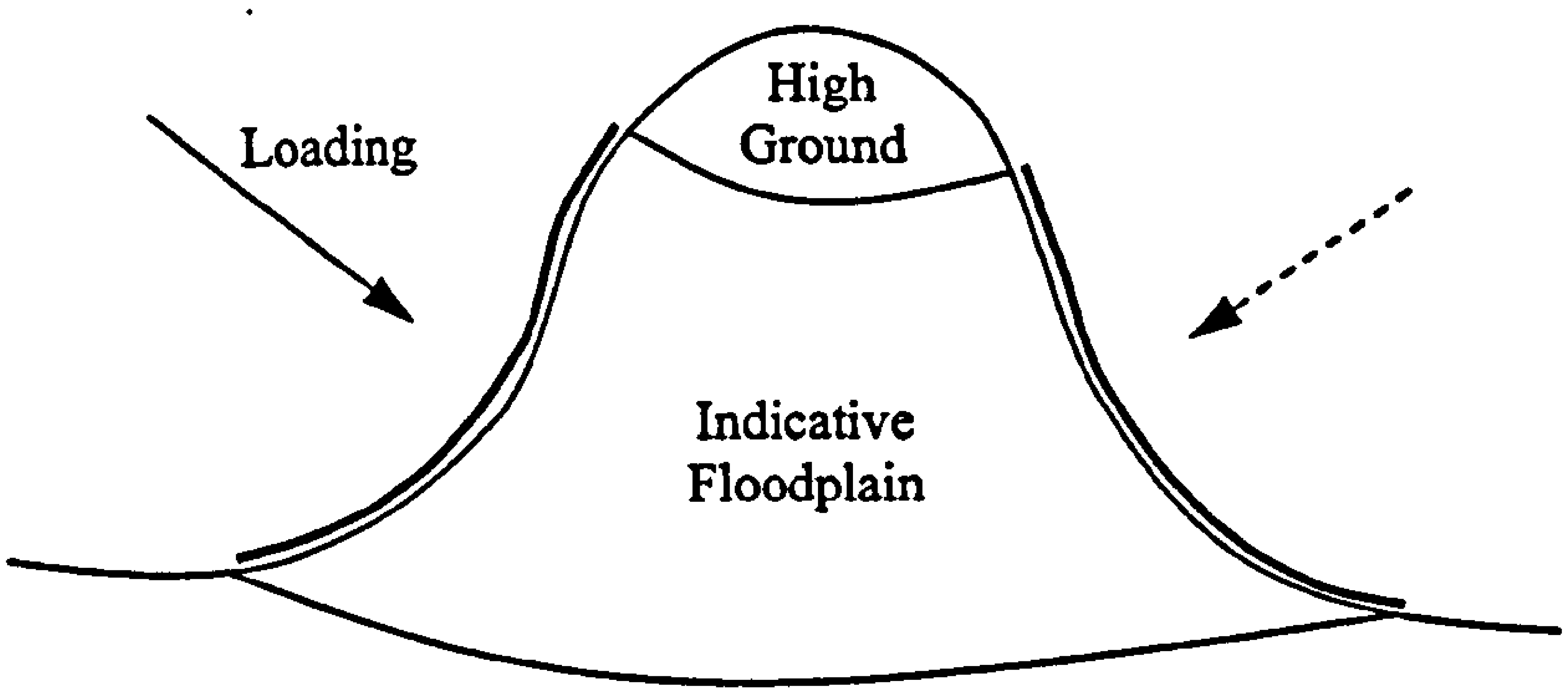


Figure 4.16 How defences of the same system, protecting the same area of coastal land can be subjected to different loadings.

4.6.5. Impact estimation

Hydrodynamic modelling offers the opportunity to improve on the high level methodology as flood depths are spatially indexed. The depth of water in the flooded area is not approximated to the same value for every building within an impact zone for a given failure scenario as it is in the high level methodology. The inundation model provides a flood outline and flood depths that account for the topography of the floodplain.

As with the high level methodology, depth-damage curves taken from the Multi-Coloured Manual (Penning-Rowsell *et al.*, 2003) are used to estimate flood damage. Social risk is measured using population numbers, the Social Vulnerability Flood Index (SVFI) (Tapsell *et al.*, 2001).

At the intermediate level it is appropriate to consider additional damages to those evaluated at the high level. Inclusion of secondary direct damages, such as transport disruption, provides an estimation of these sometimes significant economic impacts and allows transportation engineers and emergency services to incorporate flooding into their planning process.

4.6.6. Uncertainty

The uncertainty is, as at the high level, accounted for by identifying upper and lower bounds for the most uncertain quantities. For example, the uncertainty in defence fragility is represented by upper and lower bounds on the curves. Uncertainty in defence crest levels and DEM heights can also be accounted for with bounds (*eg.* LIDAR data often used to generate DEMs can have an accuracy of ±0.1m).

Uncertainty in loadings is considered by analysing a number of loadings that share the same probability of occurrence. For example, the 100 year fluvial flood event will have different flow rates depending on the storm duration. The highest and lowest river levels at each defence are established by analysing a number of the storm durations. Uncertainty in coastal loadings is considered through the modelling of a large number of combinations of wave and water level.

4.7. DETAILED LEVEL METHODOLOGY

The detailed level is, at the time of writing, at a less developed stage than the intermediate level. This level of the methodology aims to support scheme design and optimisation. The methodology involves continuous simulation of loading and system response using real or synthetic time series data. Continuous simulation offers a number of benefits that are not achievable at the intermediate level.

(1) Flood defence failure is no longer considered to be independent by considering the autocorrelation of the resistance variables.
(2) Correlation of coastal loads is considered by simulating a long time series of loading at each defence that is obtained from an offshore to nearshore transformation.
(3) The sequence of defence failure can be modelled.
(4) The duration of the loading can be modelled.
(5) Flood duration can be measured.
(6) Variation in antecedent conditions (*eg.* volume of storage reservoirs, beach levels) can be captured.
(7) Sequential flood events (and their resulting impact on the system) are modelled.

This enables the detailed appraisal of alternative interventions, including responsive strategies that react to observed system behaviour. Despite a large number of benefits, continuous simulation does have its disadvantages. These are mainly attributable to the significant increase in computing resources required. Aside from the length of time required to run each simulation, extreme events which may be of interest to the flood risk manager may not be modelled, even over a long simulation.

Defence performance can still be described by fragility curves. These curves are generated from an uncertain reliability analysis of multiple limit state functions. This is described in Chapter 5. Another approach is to adapt the reliability methods developed in the Netherlands (CUR and TAW, 1990). The applicability of these methods in the UK has been demonstrated at the Caldicot levels on the Severn Estuary (Buijs *et al.*, 2003).

A more rigorous evaluation of uncertainty is appropriate at the detailed level. In addition to considering uncertainties from loading, defence response and economic damages, model uncertainty may be measured. Aronica *et al.* (2002) have already demonstrated how inundation models can use the GLUE methodology of Binley and Beven (1995). However, this requires a large number of additional model realisations as well as good training data and may not be feasible.

4.8. SUMMARY

Chapter 2 identified risk assessment as playing a key role in the future of decision-making in flood defence management. Building on the work of Meadowcroft *et al.* (1996) a tiered risk assessment methodology has been proposed. The proposed methodology explicitly considers the defence system and uncertainty in each step of the assessment.

This Chapter has introduced this tiered approach to risk assessment of flood defence systems and identified how different resolutions of analysis can be used to support decisions at all levels within flood defence management. The *high level* methodology which is suitable to make an assessment of flood risk for the entirety of England and Wales has been described in detail. A case study of the river Parrett and Bridgwater Bay has demonstrated the applicability of the method and its potential as a tool for decision-makers. The more rigorous *intermediate* and *detailed* levels of the methodology are part of ongoing research but their methods have been outlined. Methods of improving the probabilistic description of defence failure, more suitable for a detailed level of risk assessment are introduced in Chapter 5.

Chapter 5

Improved condition characterisation

5.1. Overview

A review of flood defence management in Chapter 2 identified the need and requirements for a condition characterisation methodology. This is being driven by the need for a quantitative risk-based approach to flood management introduced in Chapter 4. Systems based risk assessment requires an assessment of the condition of the flood defences within the system. The level of analysis required for the risk assessment and consequently the condition characterisation should be appropriate for the decision being supported.

Chapter 4 introduced fragility curves that were constructed using expert judgement for broad assessment of flood. The condition characterisation methodology proposed in this chapter uses reliability methods to establish fragility curves. This approach allows explicit consideration of multiple failure modes and bounds of failure for a structure can be established. Traditional reliability methods have been adapted to allow expert judgement which is so often a part of condition characterisation to be represented by describing failure parameters using membership functions. This method can be adapted to monitor the loss in structural performance by modifying appropriate parameters. The use of models to predict the behaviour of these parameters also allows the engineer to predict future losses in performance which is critical for long term strategic planning. Whilst the methodology can be used to identify the data most likely to reduce the uncertainty associated with the condition characterisation and therefore target resources more efficiently, the emphasis of the methodology is not on improving data acquisition methods, but rather on maximising the use of available information. The different aspects of the condition characterisation methodology are described and supported using examples.

5.2. Risk based condition characterisation

5.2.1. Key requirements

A high level target for the Environment Agency of England and Wales requires that flood risk assessments are to be made on a national scale (DEFRA, 1999). Decision-makers have for many years been making investment choices based on a qualitative comparison of risk (*eg*. Burgess and Reeve, 1994) which only provide comparative measures of risk. To provide a quantitative assessment of risk, a quantitative measure of the probability of the hazard, in this case the probability of defence failure is required. A probabilistic condition characterisation provides this.

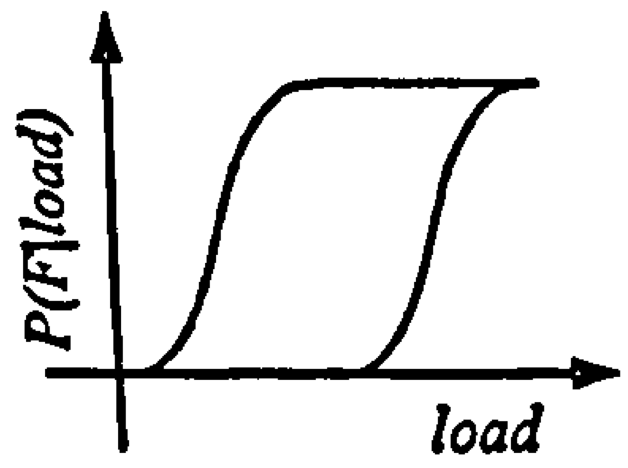
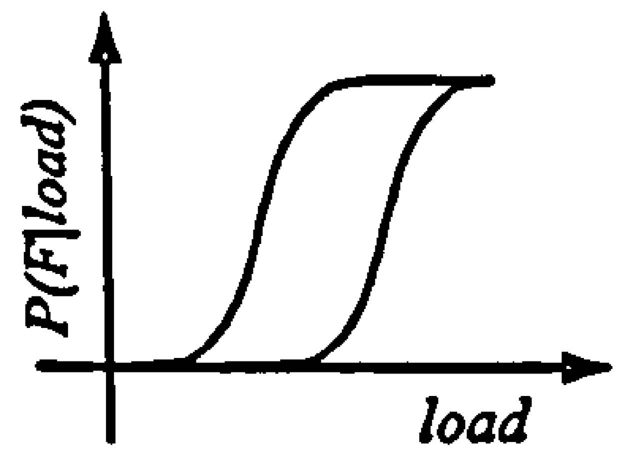
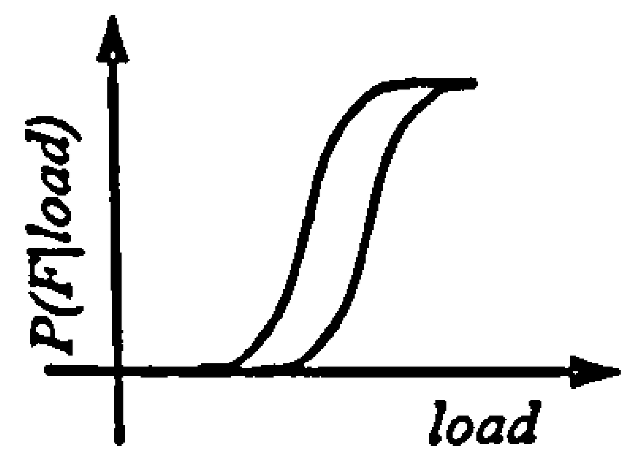
Chapter 4 introduced a tiered risk assessment methodology that recognised the need for a level of analysis appropriate to the decision being supported. Naturally this should reflect the level of analysis used to perform the condition characterisation. Table 5.1 shows how some of the decisions supported by a risk assessment require more accurate assessments of flood risk and consequently condition characterisation.

Risk-based asset management of coastal defences requires an assessment of the probability of failure of each element in the coastal defence system. However, engineers engaged in condition characterisation do not usually have sufficient information to generate precise failure probabilities. This is because the information available to them (Section 5.2.3) may be vague or incomplete and perhaps of questionable dependability (for example if it is based on the judgement of a single inspecting engineer). A condition characterisation is therefore required that can consider all evidence on defence performance regardless of its format and explicitly consider the uncertainty associated with this evidence.

Moreover, to generate a probability of failure not only requires an assessment of the defence strength (which is the topic of condition characterisation) but also an assessment of the loading conditions. Loading conditions are generated by hydraulic and statistical analysis that is conducted by other experts and may not be available to the inspecting engineer when they are conducting their inspection. The frequency with which the analysis of these loads is updated does not necessarily coincide with the frequency of defence inspection. Therefore, it is preferable that the assessment of defence condition should be separated from the assessment of loading. This is achieved by characterising the defence strength with a fragility function (Section 5.2.2).

A thorough condition characterisation should specifically identify the failure modes of the defence under consideration and make use of existing generic knowledge of the variables that influence those failure modes. This generic knowledge is customarily expressed in terms of algebraic relationships between the load and strength of the defences which are usually presented as design formulae. Parameter values should be used that are appropriate to the defence condition. Design formulae address the defence when it is in its as-built condition, so without adjustment are inappropriate for the assessment of degraded defences. The effect that degradation will have on the state variables describing the defence strength needs to be identified. The variables and hence defence performance are adjusted appropriately.

Table 5.1 Showing increasing degree of analysis and accuracy to support different decisions

Decisions to be supported		Uncertainty of condition characterisation
National assessment of flood risk Prioritisation of expenditure Prioritisation of maintenance	Increasing level of analysis	
Flood defence strategy planning Regulation of development		
Scheme appraisal and optimisation		

5.2.2. Expressing defence performance with fragility functions

The fragility (Casciati and Faravelli, 1990) of a structure is the probability of failure, conditional on a specific loading, L. If the failure of a structure is described by a limit state function Z such that $Z \leq 0$ represents system failure and $Z > 0$ represents the not failed condition, then the fragility function $F_R(L) = P(Z \leq 0 \mid L)$. A fragility curve is a plot of the conditional probability of failure of the structure given a complete range of loadings. Fragility curves generated by expert judgement were introduced in Chapter 4 to describe the probability of defences breaching and overtopping.

In Figure 5.1 the load under consideration is the significant wave height H_s. The fragility curve therefore provides a complete probabilistic description of the strength of the structure under the full range of loading conditions. Coastal defences are usually subject to more than one type of loading, most typically both wave height, H_s, and water level, W. Thus in general the fragility will be a function of several loading variables $L_1 \ldots L_n$.

The fragility function can subsequently be combined with the loading distribution to generate a probability of defence failure, $P(Z \leq 0)$.

$$P(Z \leq 0) = \int_0^{\infty} F_R(l)dl \qquad (5.1)$$

For the case illustrated in Figure 5.1, suppose that the significant wave height is represented by a Gumbel distribution with parameters $\alpha = 2.5$ and $\xi = 1.7$, then integrating (numerically) the loading distribution with the fragility function gives an annual probability of defence failure of 0.012.

Previously in Chapter 4, fragility curves to describe a defence's proneness to overtopping and breaching were constructed using expert judgement to estimate conditional failure probabilities for a few given loads. Whilst this acts as an important first step onto the risk management ladder in information poor situations, a reliability analysis based on a number of failure modes provides a more accurate and auditable assessment of structural performance.

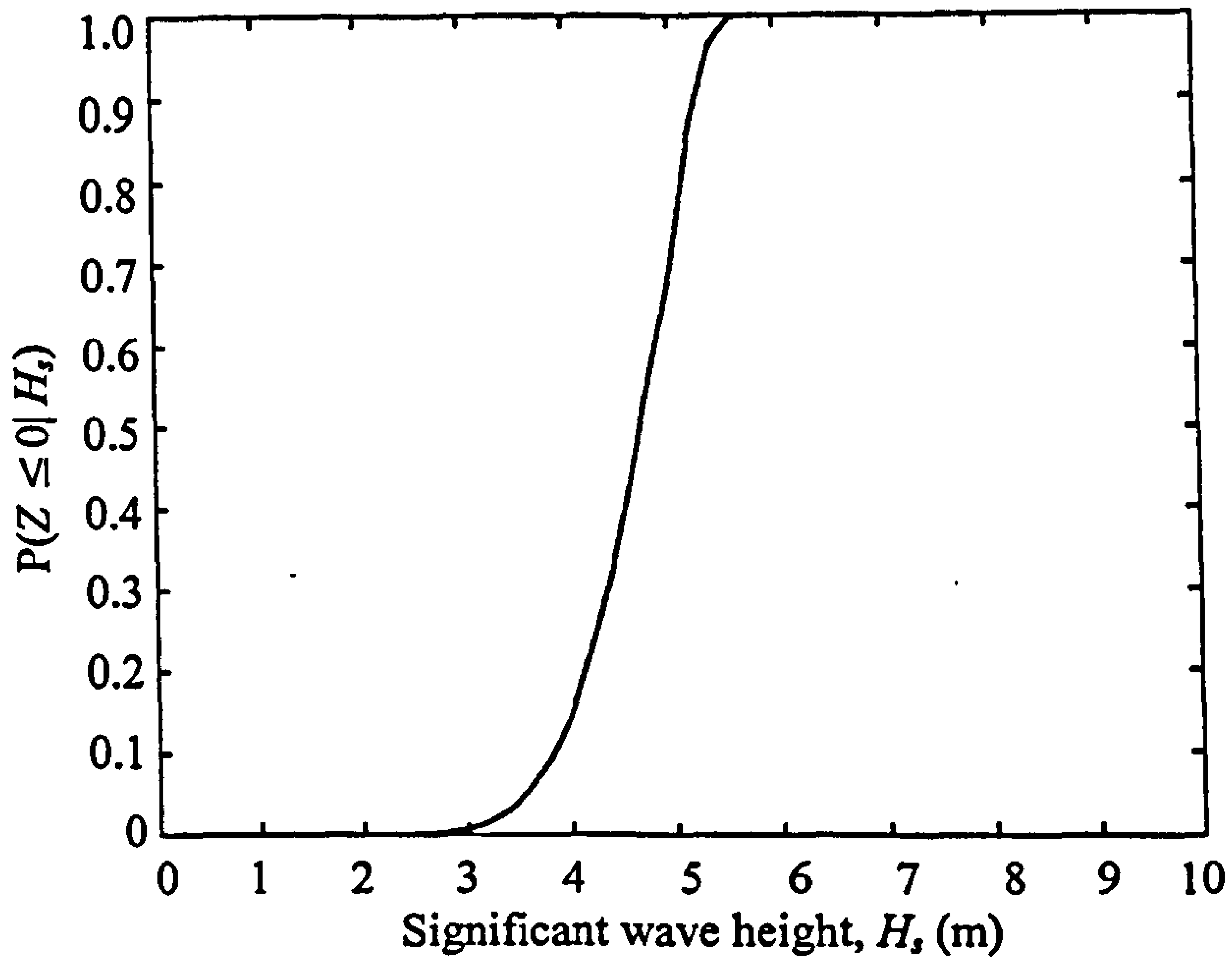


Figure 5.1 A typical fragility curve

5.2.3. Condition characterisation evidence

Information on defence condition appears in a range of formats, from precise measurements to vague expert judgements. Table 5.2 includes some examples of the types of evidence that may be available to an engineer engaged in a condition assessment.

Table 5.2 Examples of condition characterisation support evidence

Source of evidence	Evidence format	Example of evidence	Use of evidence
Expert judgement	Linguistic	Structure is 'safe'	General assessment of degradation
Expert judgement	Integer	Overall rating from 1-5	General assessment of degradation
Visual observation	Linguistic	Crest 'uneven'	Observations of individual variables
Measured observation	Precise numerical value	Settlement of crest by 0.5,	Precise failure mode analysis
Population of measurements	Probability distribution	Mean armour size = 1.5m and variance = 0.1m	Probabilistic analysis
Generic knowledge	Range of values	Soil is loose and sandy therefore we can estimate: $25° < \phi_{cv}' < 35°$	Vague failure mode analysis

The Bayesian school of probability suggests that all of these types of uncertain information should be mapped onto a precise probability distributions (Lindley, 1971). The approach adopted here is based on the principle that inherent uncertainties in nature (for example due to the unpredictability of future wave conditions) should be separated from epistemic uncertainties (for example of soil strength at a given defence cross-section) as recommended by Hofer (1996). Uncertainty in condition characterisation is dominated by these epistemic uncertainties, which can be represented by making imprecise mathematical statements, either in the form of interval bounds on an unknown value, or, more generally, by constructing a fuzzy set over an unknown value. A fuzzy set can be thought of as a 'fuzzification' of an interval measurement in which the interval bounds are softened by assigning a membership function, μ, (on a [0,1] scale) to values near the bounds of the interval. The membership function can be thought of as a measure of the *possibility* of encountering a given value (see Section 3.2.4 for a more thorough description of possibility and membership functions).

For example, the rock diameter, D, of a revetment may be expressed in a range of formats. A newly built structure may have precise information available from design specification and construction records, in which case rock diameter may be presented as a probability distribution such as a normal distribution with μ = 1.5m, σ = 0.1m. For existing structures precise distributions of rock diameter will seldom be available, in which case a small sample of measurements could be used to estimate bounds on the rock size, for example D=1.5-2.0m. This could be generalised to a fuzzy set, indicating the possibility of a particular value for D. Figure 5.2 shows an example of a trapezoidal fuzzy set showing that the most *possible* values for D are between 1.6m and 1.9m, but with a decreasing possibility it could be as little as 1.5m or as great as 2.0m. Newberry *et al.* (2002) demonstrated the capability of engineers to rapidly estimate with a high degree of accuracy characteristics of rock armour and it is this sort of expertise that can be used to assign fuzzy sets through observation. This technique should be supported by a number of measurements, identification of the largest and smallest rocks can be used to place upper and lower bounds on the distribution. Visual identification of the most common rock size allows bounds to be placed on the most possible rock diameter, hence fully defining the fuzzy set.

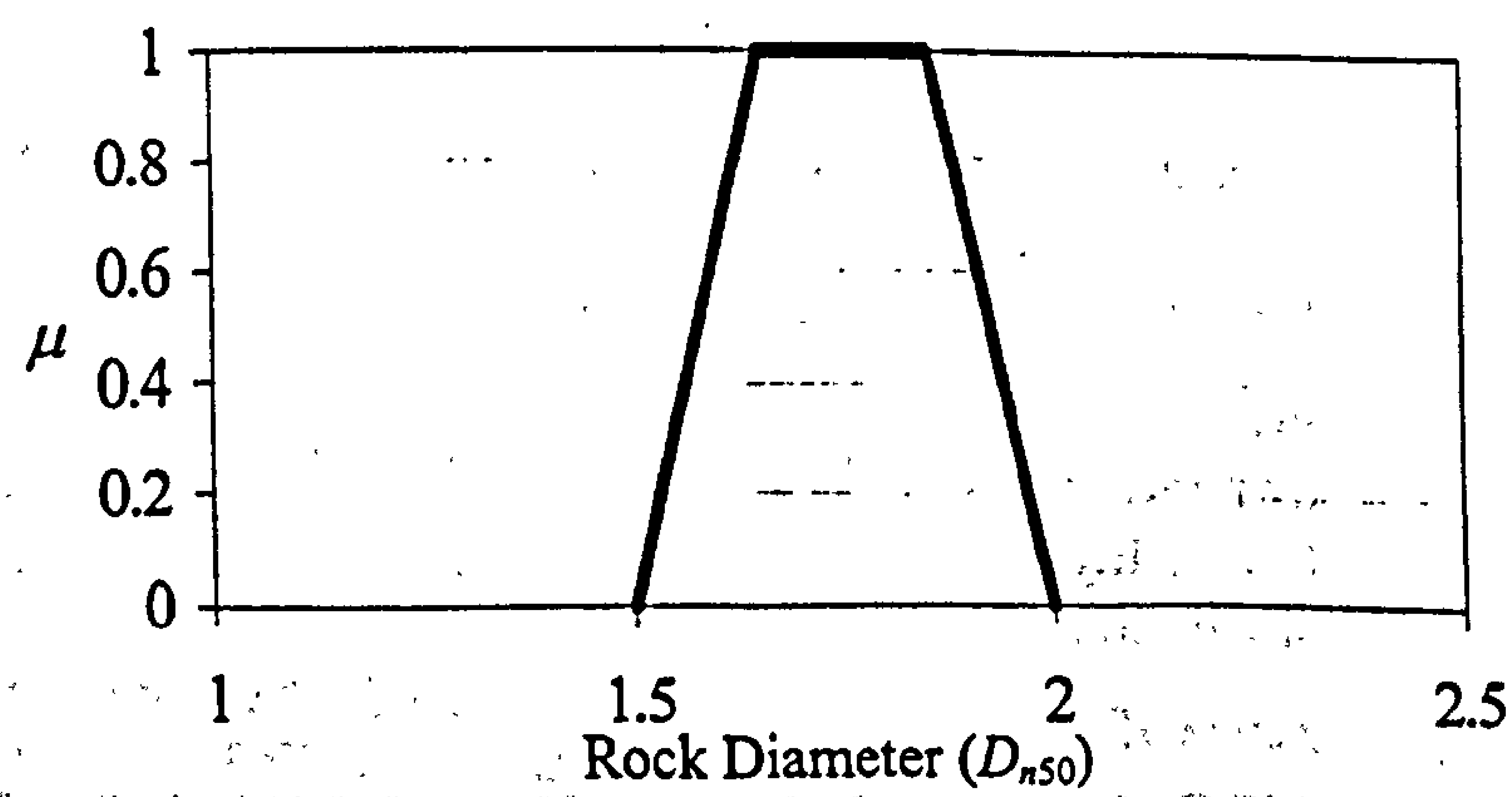


Figure 5.2 An example of a fuzzy set defining the possibility of values for the rock diameter, D

Imprecise parameters can be used in the assessment of the fragility function, in which case, rather than generating a single fragility curve, (fuzzy) bounds on a family of fragility curves are generated. Entering multiple parameters as fuzzy or interval values or increasing the bounds on these values results in an increase in uncertainty associated with the structure's performance. This has the effect of widening the bounds on the fragility curves. This imprecise fragility function can then be integrated, using Equation 5.1, with the loading distribution to generate interval bounds on the probability of failure. This is illustrated with supporting examples in the following sections.

5.3. APPLICATION

5.3.1. Single failure mode

The condition characterisation methodology is described using an example. First a fragility curve representing the proneness to failure of the rock armour revetment shown in Figure 5.3 is calculated. The analysis is then repeated using different types of vague information about the dyke in order to generate an imprecise fragility function.

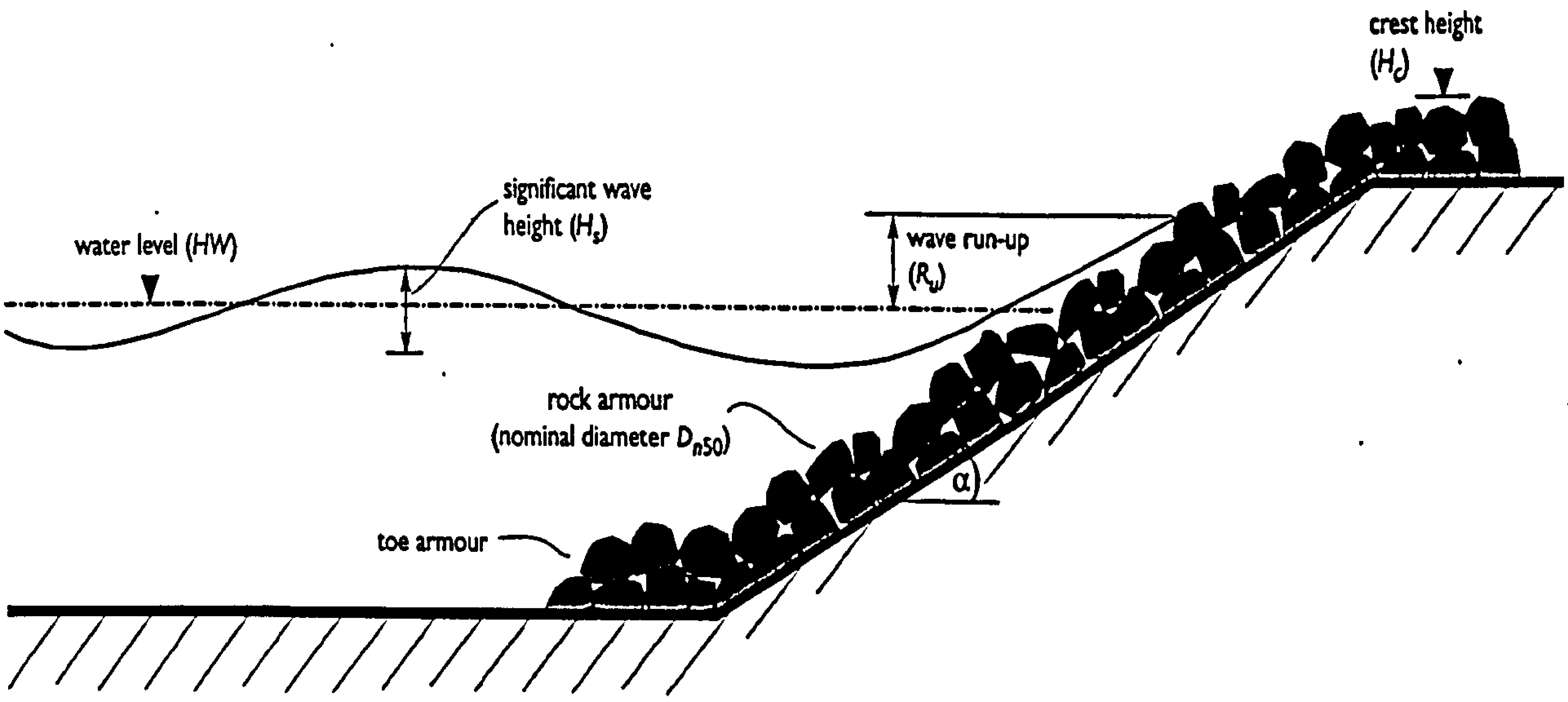


Figure 5.3 Definition of variables for typical rock armour revetted dyke

The probability of revetment failure can be estimated using Van der Meer's formula (Van der Meer, 1988) for armour stability and a First Order Reliability Method (Melchers, 1995). The limit state functions from Van der Meer's formula are given by Equations 5.2 and 5.3.

$$Z = 6.2S_d^{0.2}P^{0.18}\Delta D_{n50}(\cot\alpha)^{0.5}s_m^{0.25}N^{-0.1} - H_s \tag{5.2}$$

for plunging waves, and

$$Z = S_d^{0.2}P^{-0.13}\Delta D_{n50}(\cot\alpha)^{(0.5-P)}s_m^{-0.5P}N^{-0.1} - H_s \tag{5.3}$$

for surging waves, where P is the permeability factor; $\Delta=\rho_{rock}/\rho_{water}-1$; D_{n50} is the nominal rock diameter; α is the revetment slope; s_m is the mean wave steepness; N is the number of waves attacking the structure and S_d is the damage number. This damage corresponds to a non-dimensional eroded area which is defined by Broderick and Ahrens (1982) as:

$$S_d = \frac{A_e}{D_{n50}^2} \tag{5.4}$$

where A_e is the area of erosion around the still water level (Figure 5.4).

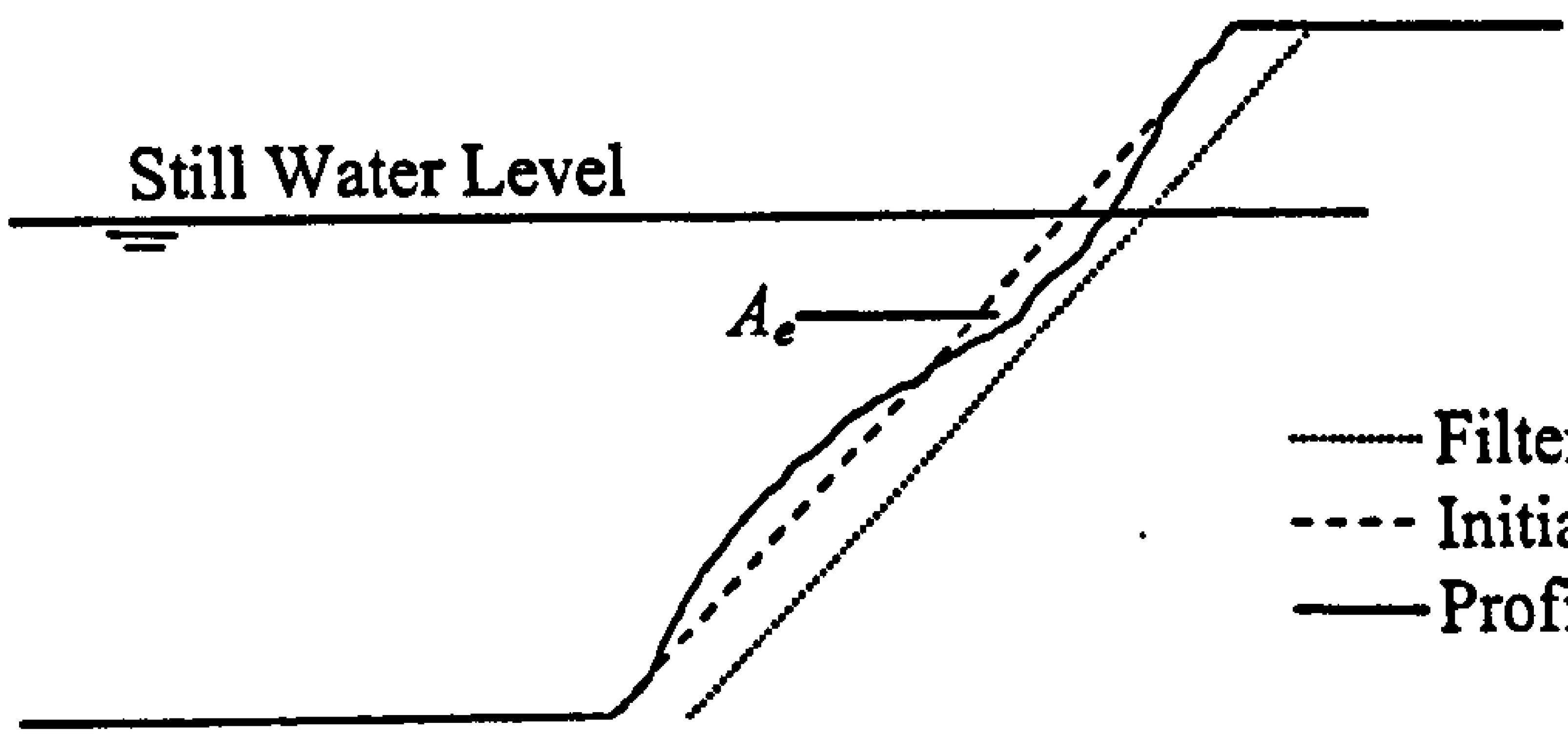
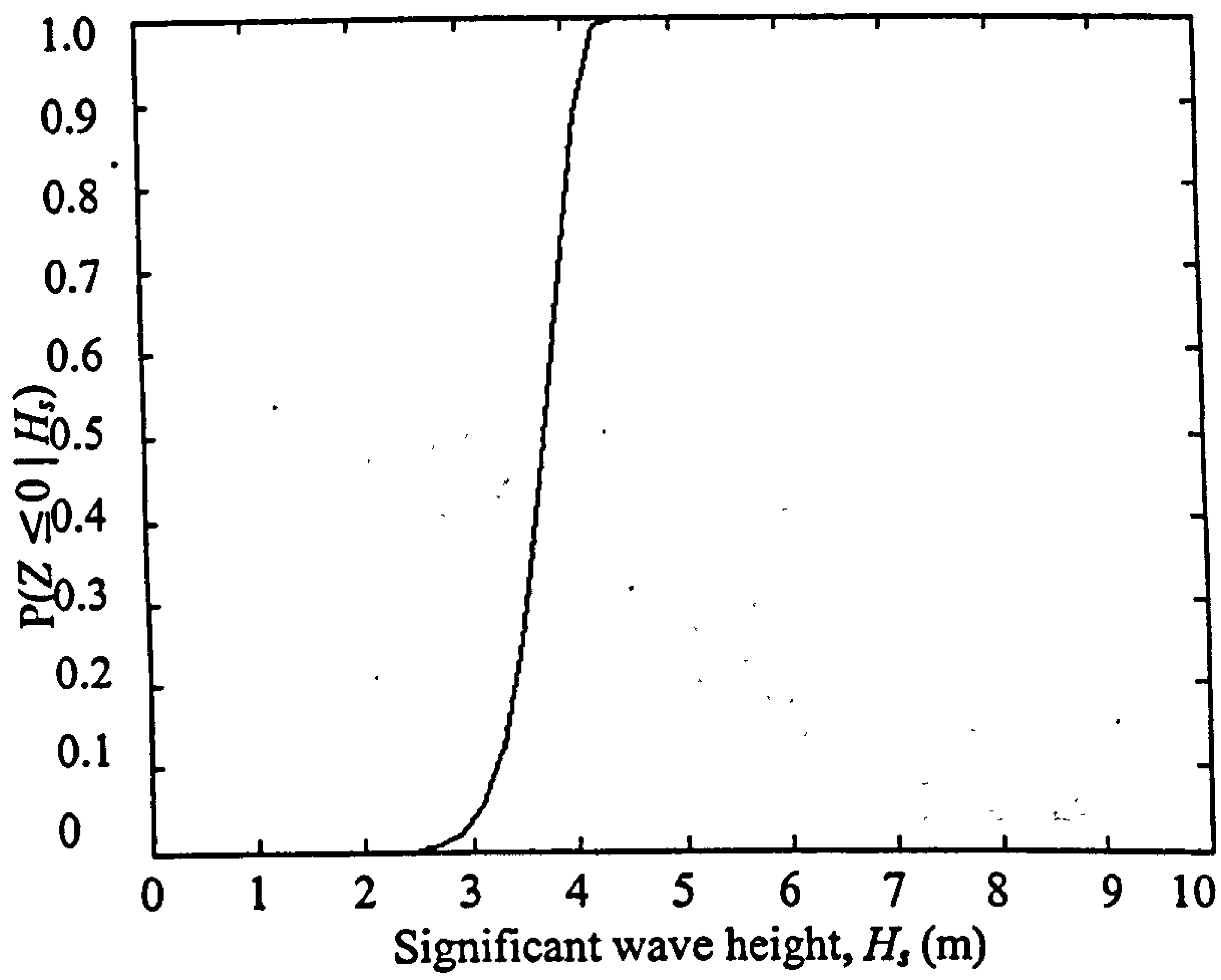


Figure 5.4 Definition of erosion area of a revetment

In Equations 5.2 and 5.3 the loading (L) is H_s. By conducting the reliability analysis over a range of wave loadings the fragility curve shown in Figure 5.5 was generated.



Probabilistic parameters (normally distributed):

α $\mu = 25°; \sigma = 2.5°$

D_{n50} $\mu = 1.5\text{m}; \sigma = 0.1\text{m}$

$\mu = 1.6; \sigma = 0.1$

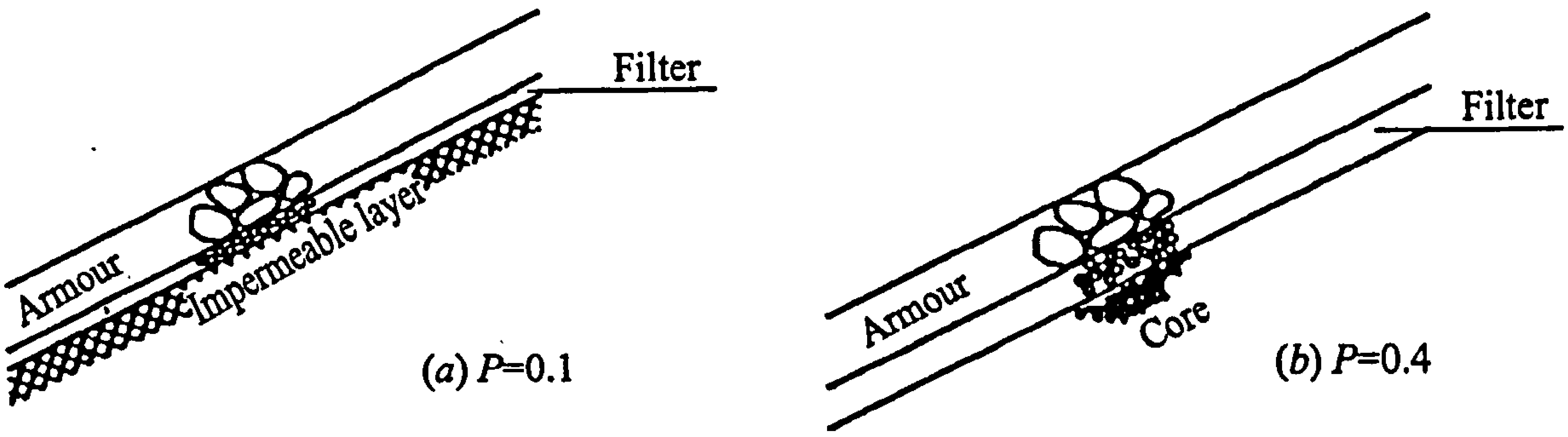
Other parameters:

P	0.1
N	3000
s_m	0.065
S_d	2

Figure 5.5 Fragility curve of a revetment's proneness to failure using Van Der Meer's formula to determine revetment stability

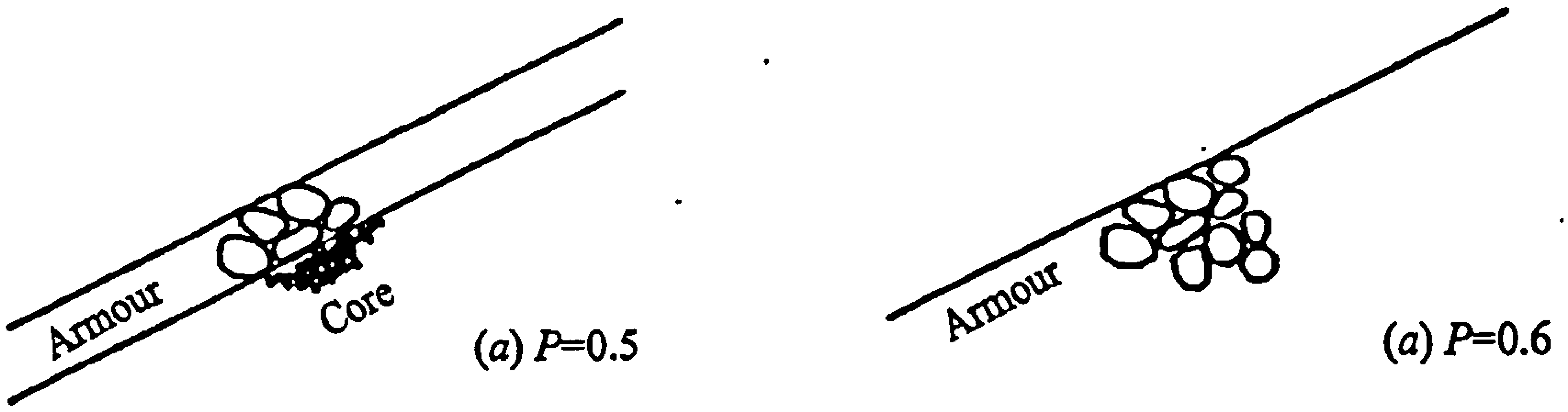
5.3.2. Incorporating imprecise information

As described in Section 5.2.3 a fuzzy set can be used to capture the uncertainty in the diameter of rock armour. Uncertainty in other parameters can also be captured by this means. This section considers the permeability of a structure, P, which is assessed based on the similarity of the structure with some prototypical pictures and the diameter of core, filter and revetment material (Figure 5.6). Estimation of P is therefore always, to some degree, subjective.



- Armour layer thickness = $2D_{n50}$A
- Filter layer thickness = $0.5D_{n50}$A
- D_{n50}A / D_{n50}F=4.5

- Armour layer thickness = $2D_{n50}$A
- Filter layer thickness = $1.5D_{n50}$A
- D_{n50}A / D_{n50}F=2, D_{n50}F / D_{n50}C=4



- Armour layer thickness = $2D_{n50}$A
- D_{n50}A / D_{n50}C=3.2

- No filter
- No core

Figure 5.6 Permeability factor of revetments for (a) P=0.1, (b) 0.4, (c) 0.5 and (d) 0.6

A newly built structure may have precise information available from design specification and construction records, in which the permeability factor, P, is defined as being 0.1. Consider now the situation for existing structures, in which the filter layer may have clogged up, or the exact nature of the underlayers is unknown. An exact figure for P will be difficult to obtain and it may be economically unviable to perform a detailed investigative survey. In this case bounds based on brief visual observations or previous experience may be assigned to define the permeability, for example, P= 0.1-0.4. This information can be used to calculate bounds on the fragility curve (Figure 5.7) and can also be thought of as a sensitivity test. Integration, using Equation 5.1, with the loading distribution generates interval bounds on the probability of failure.

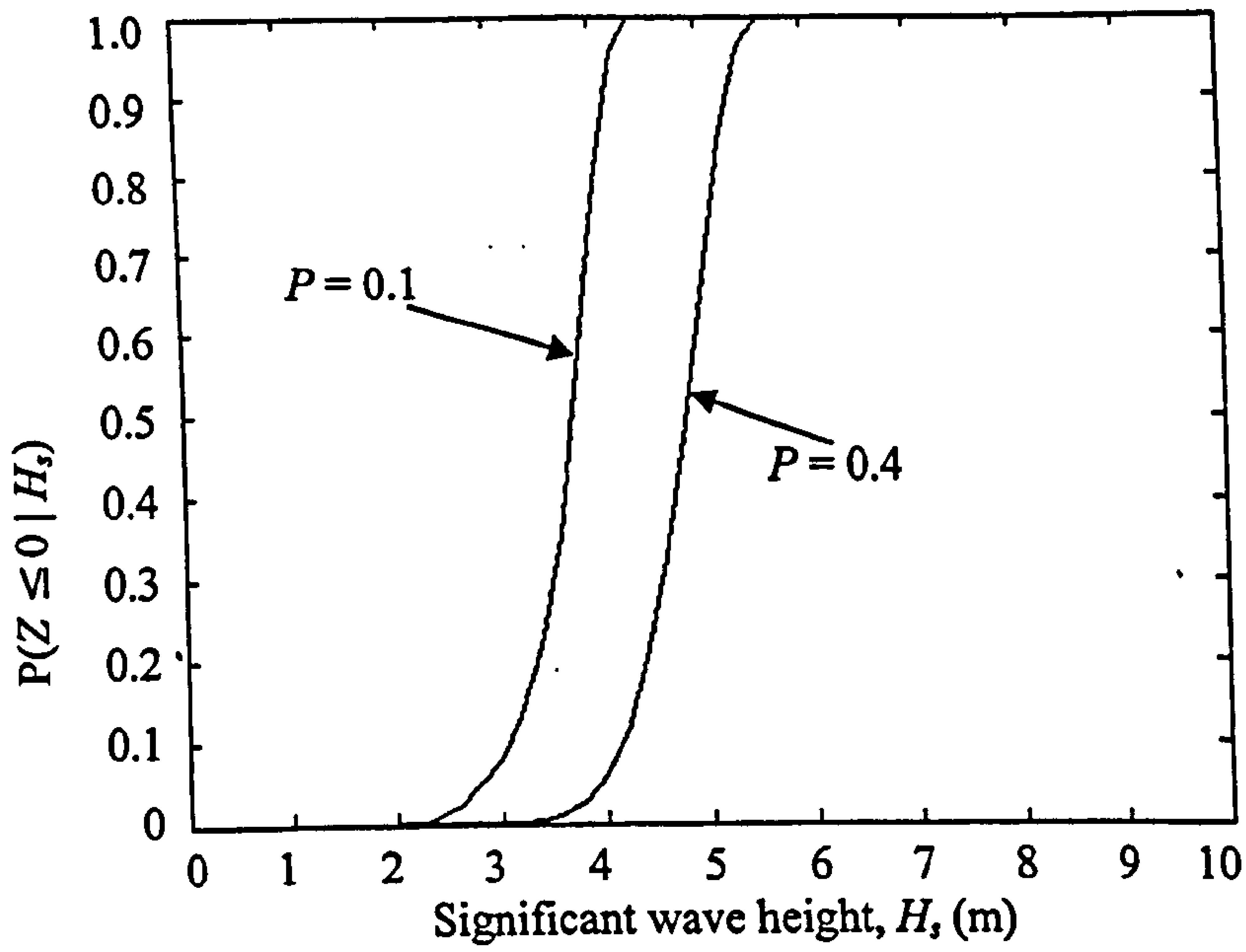


Figure 5.7 Imprecise fragility curve of Van der Meer's equation ($P \in [0.1, 0.4]$). Other values as in Figure 5.5)

Using the fuzzy set defined in Figure 5.8 instead of an interval to define *P*, a more informative description of the defence fragility can be obtained. This trapezoidal fuzzy set shows the most possible values for *P* are between 0.2 and 0.5, but with a decreasing possibility *P* could be as little as 0.1 or as great as 0.6. This produces a family of fragility curves, shown in Figure 5.9. The area between the middle two curves represents the most possible failure space, with decreasing possibility of occurrence to the outer curves. This can be integrated with the loading to provide fuzzy probability bounds. In Figure 5.10 four of the variables that determine revetment failure have been represented as fuzzy sets, with a consequent increase in uncertainty in the fragility curves.

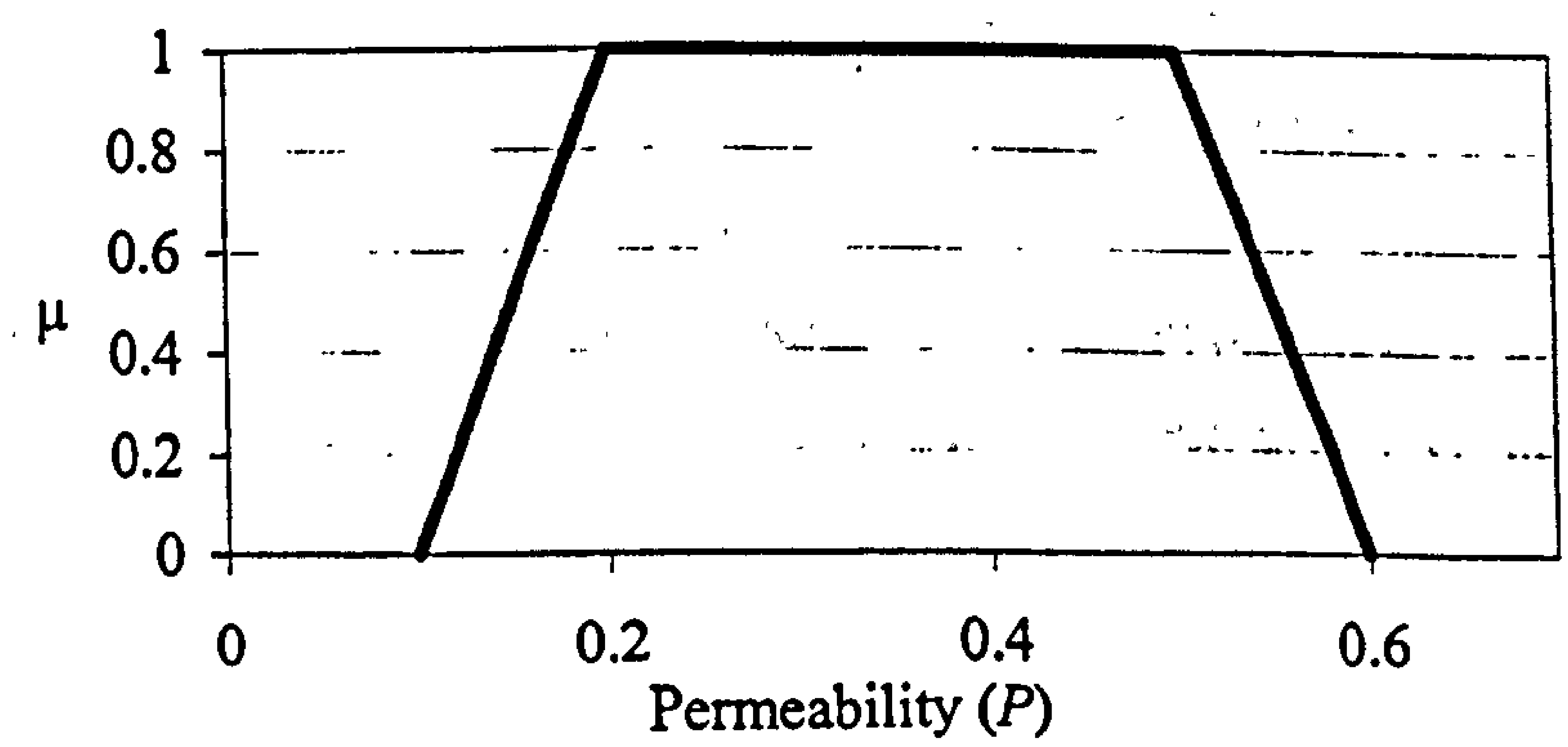


Figure 5.8 The fuzzy set defining the possibility of values for P

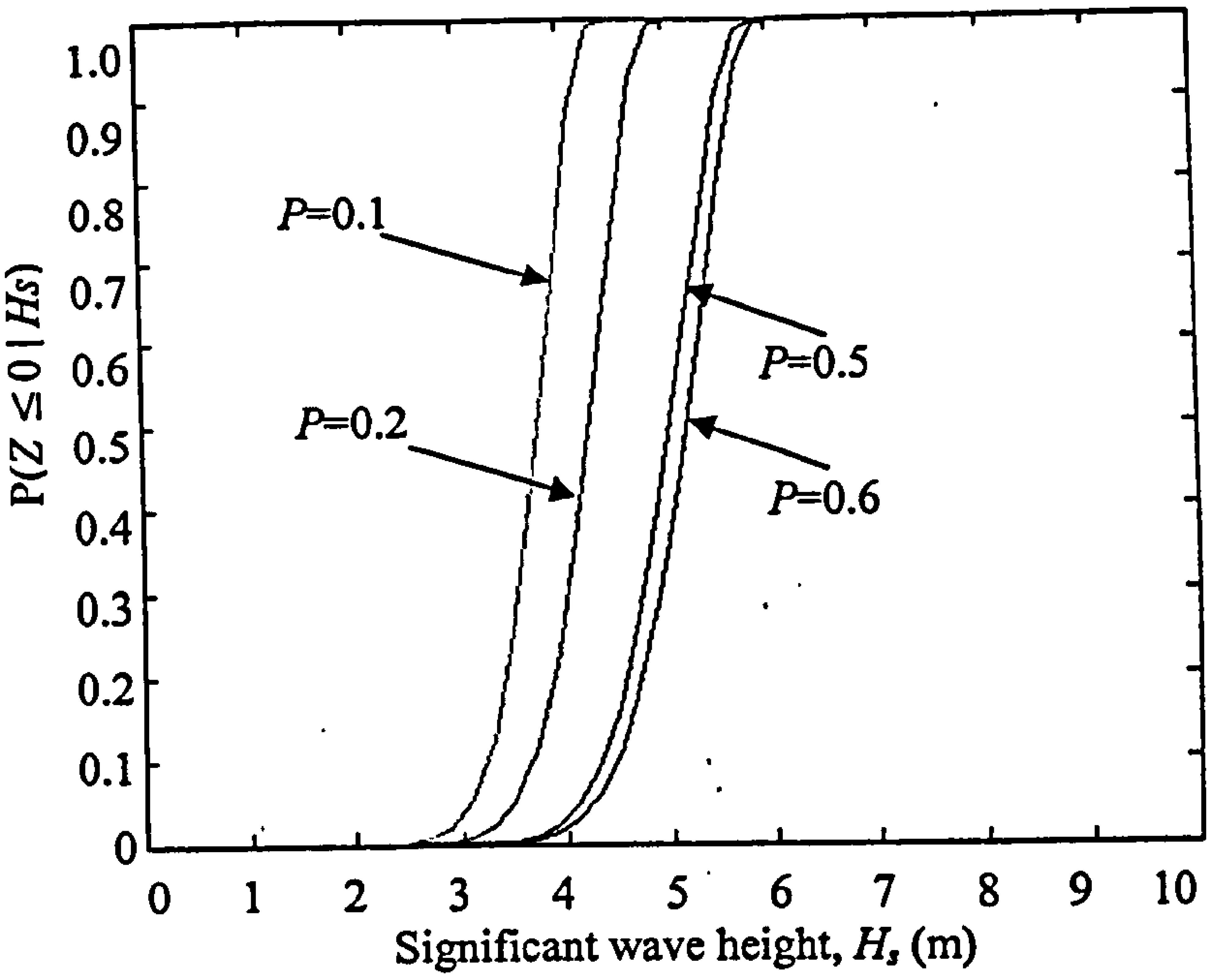


Figure 5.9 Family of fragility curves with P defined by the fuzzy set in Figure 5.8 (all other values as in Figure 5.5).

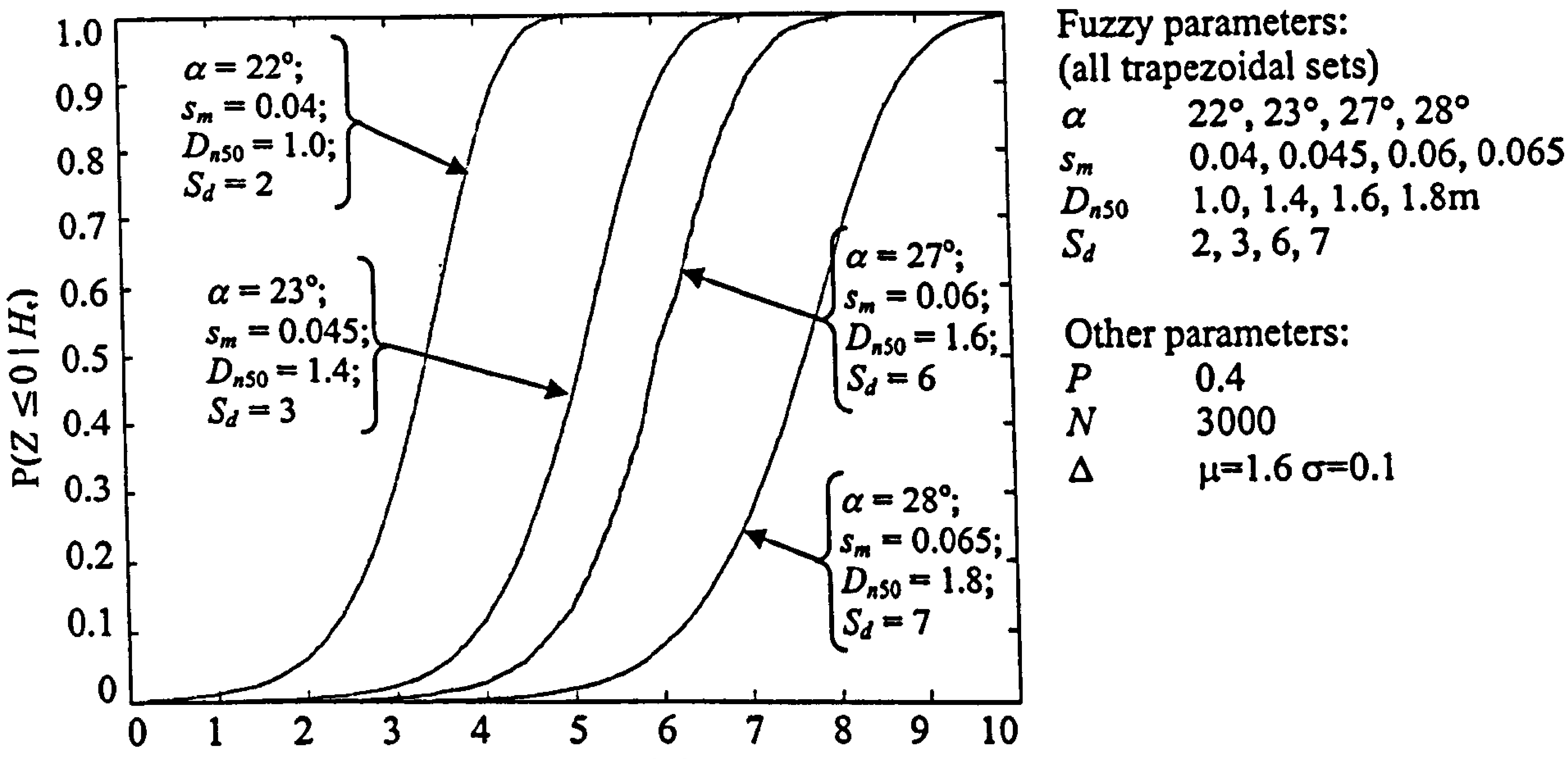


Figure 5.10 A family of fragility curves generated as a result of defining multiple parameters as fuzzy sets

5.3.3. Multiple failure modes

Two further failure modes, run-up and toe stability were then included to the example introduced above. The limit state for run-up is defined by Vrijling (1993):

$$Z = H_c - HW - R_{u2\%} - H_s \tag{5.5}$$

where H_c is the height of the dike crest, HW is the water level, and $R_{u2\%}$ is a measure of the run-up for the highest waves and is defined as:

$$R_{u2\%}/H_s = \tan\alpha/\sqrt{s_m} \tag{5.6}$$

Toe stability is defined by Van der Meer *et al.* (1995).

$$Z = \left(0.24\frac{h_t}{D_{n50}} + 1.6\right)N_{od}^{0.15}\Delta D_{n50} - H_s \tag{5.7}$$

where h_t is the toe depth, N_{od} is the damage factor (which ranges from 0.5 at the start of damage, 4.0 at failure). The original parameters used to generate Figure 5.5 were used with a crest height of 10.5m and the toe at 3m below a fixed water level.

Ditlevsen's (1979b) second order bounds (defined in Chapter 3) were calculated for these three failure modes to generate two fragility curves that define the upper and lower bounds of conditional failure probability of the breakwater. These bounds are plotted in Figure 5.11.

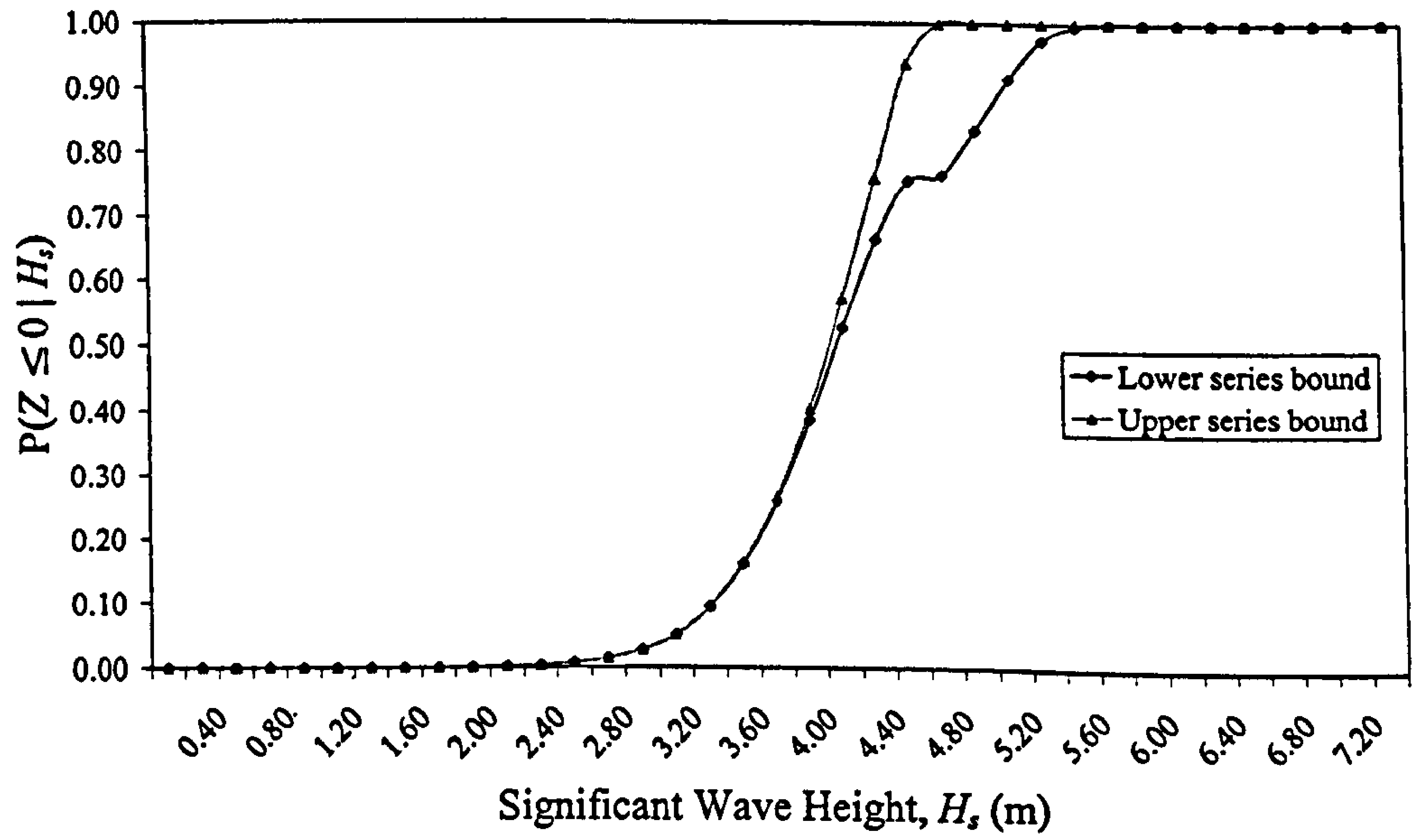


Figure 5.11 Second order series system failure bounds for three failure modes

5.4. PERFORMANCE LOSS FROM STRUCTURAL DEGRADATION

Degradation in defence condition influences the probability of failure by changing variables that determine the defence fragility. For example settlement influences the crest level, which is critical in overtopping calculations. Erosion of the dyke crest and back slope influences the overtopping threshold value for failure. Analysis of deteriorated defences therefore involves identification of the defence variables that have been influenced by deterioration, and, assessment of the effect that deterioration has had on the value of those variables. Being able to monitor and predict degradation is important for decision-making. In some cases it will be possible to measure the change in a variable due to degradation (for example change in the crest level due to settlement), in other cases this measurement may only be approximate (for example the degree of clogging of a filter layer). This results in the movement of the fragility curve, which represents a loss in structural performance. Table 5.3 lists some examples of variables that can be monitored to assess the degree of degradation.

Design formulae usually address the defence when it is in its as-built condition, so without adjustment are inappropriate for the assessment of degraded defences. The effect that degradation will have on the state variables describing the defence strength needs to be identified and the variables adjusted appropriately.

Table 5.3 Variables that can be monitored to assess degradation

Failure mode	Variables to monitor
Revetment stability	Damage number (*eg.* S_d)
	Armour layer thickness
	Armour stone mass
	Filter layer permeability
Overtopping	Crest level
	Roughness
Geotechnical stability	Pore pressures
	Permeability
	Geometry
Corrosion	Reduction in section modulus
Beach/dune erosion	Geometry

Some limit state functions (for example Van der Meer's formula) have a damage parameter within their formulae and so a loss in performance can be represented by altering this variable. For example, Equation 5.2 can be rearranged to calculate the damage number:

$$S_d = \sqrt{N}\left(\frac{H_s\sqrt{\xi_m}}{6.2P^{0.18}\Delta D_{n50}}\right)^5 \tag{5.8}$$

where ξ_m is equal to $\tan\alpha / s_m^{0.5}$. This can be used to predict damage from continuous wave loading (Van der Meer, 1986 and Melby and Kobayashi, 1998, 2000) by using:

$$S_{d(i+1)} = S_{d(i)} + \sqrt{\max[N_{(i+1)} - N_{Sd}, 0]}\left(\frac{H_{s(i+1)}\sqrt{\xi_{m(i+1)}}}{6.2P^{0.18}\Delta D_{n50}}\right)^5 \tag{5.9}$$

where $N_{(i+1)}$ represents the number of waves of size $H_{s(i+1)}$ in the current storm and N_{Sd} is the number of waves of this storm required to reach the current damage level of $Sd_{(i)}$. Clearly the rate of damage progression will slow as the revetment is subjected to more loads. This is because as S_d increases, so does the required number of waves to cause that damage, N_{Sd}. Therefore to progress the damage, H_s needs to be relatively large compared to previously experienced events or that there are a greater number of waves, $N_{(i+1)}$, for a given storm. In physical terms, this may be a result of the re-profiling of the breakwater cross-section.

Figure 5.12 illustrates two graphs, the upper plots the hourly time series of wave heights to which the revetment is subjected, the lower shows the increase in damage parameter over this time for three armour stone sizes, D_{n50} = 1.0m, 1.2m and 1.5m (other variables as in Figure 5.5) where an S_d value of 12 represents failure. Pozueta *et al.* (2002) have recently used a similar method as a design tool to optimise the life of a breakwater, in which the random nature of the loading was accounted for by repeating the simulation a number of times with randomly generated time series (as opposed to the measured series used in Figure 5.12).

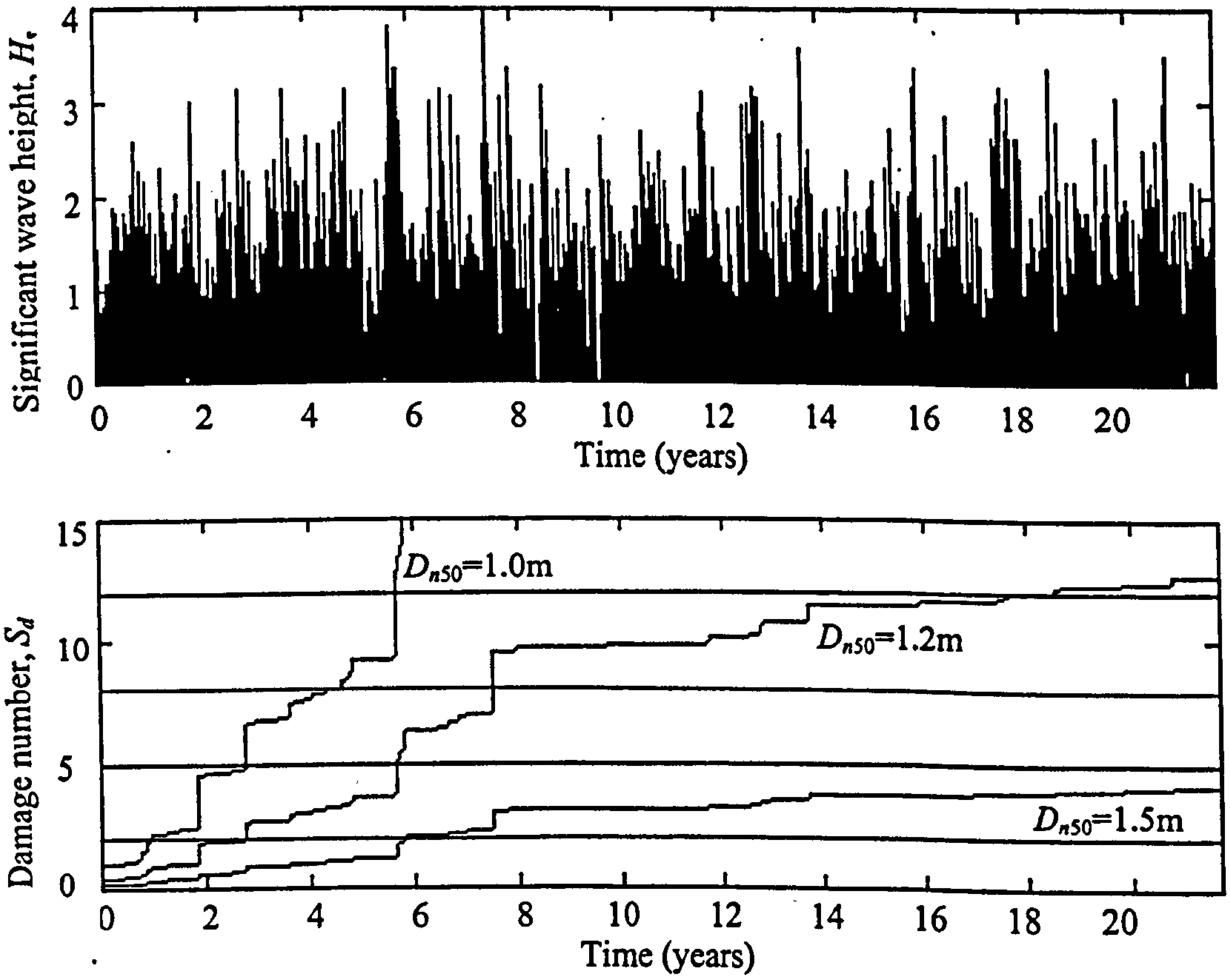


Figure 5.12 Increasing damage as a result of wave action on a revetment (S_d=12 at total failure)

Many limit state functions do not contain a damage parameter. Consider, for example, the analysis of an existing, deteriorating sheet pile wall seawall. The thickness of the wall is reducing due to corrosion, whilst the beach level fluctuates (Figure 5.13). Suppose that the section modulus S = 600cm^3, the ultimate stress of the steel σ_n~N(150,20) N/mm^2 and the beach depth d~N(7,0.5)m.

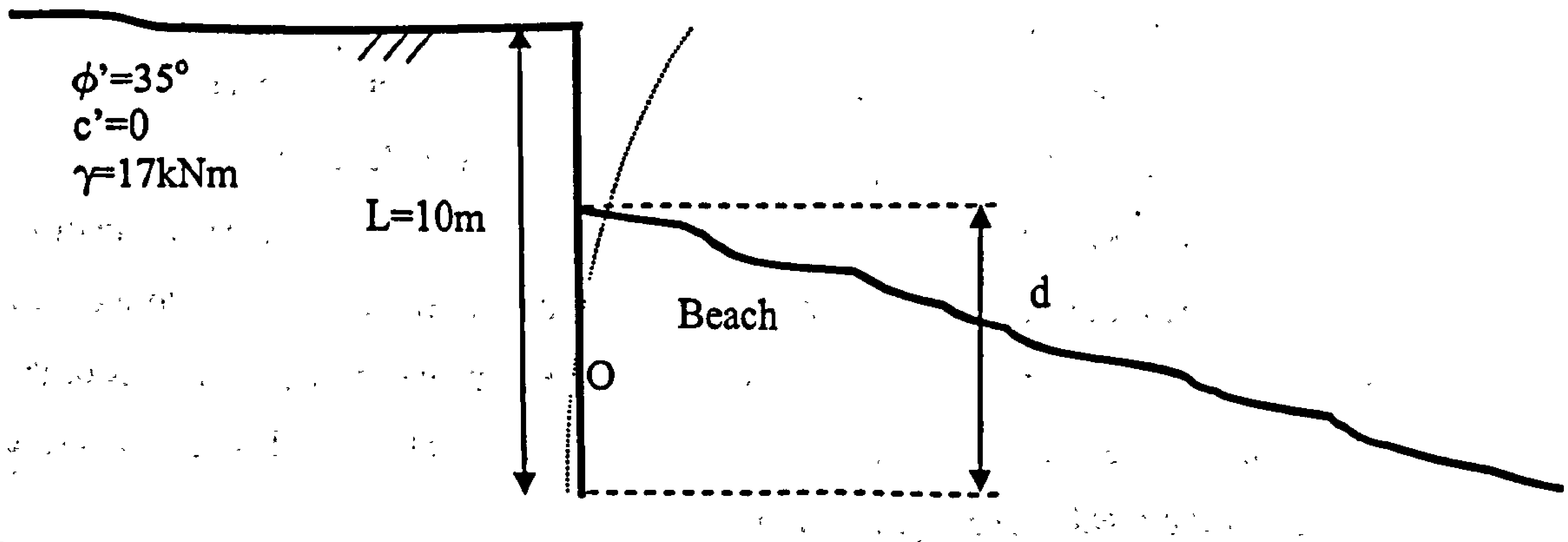


Figure 5.13 Definition of variable for sheet pile retaining wall

The limit state function for failure by rotation is defined as the point at which the passive force, P_p, exceeds the active force, P_a, about the point of rotation, O:

$$Z = P_p \times M_p - P_a \times M_a$$

where M_p and M_a are the moment arms of the passive and active force about point O. The moment capacity, M_c, of the pile is given by design guides (such as Corus Group, 2001) and the limit state function is given by:

$$Z = M_c - P_a \times M_a \tag{5.10}$$

More details on sheet pile design and geotechnical failure are given in Appendix D. Application of a first order reliability analysis (described in Section 3.4.2) generates failure probabilities of 0.019 for failure by rotation at the toe and 2.3 x 10^{-4} for failure due to insufficient moment capacity.

Frequently in England and Wales, the toe depths of structures are uncertain (Reeve and Burgess, 1994), therefore to use a precise failure probability is inappropriate. However, an engineer can use their judgement to make an assessment of the likely depth of toe based on evidence from design drawings of similar structures. A fuzzy set can be used to represent this judgement. Figure 5.14 defines a fuzzy set suggesting the pile has a maximum depth that lies between 8 and 12 metres, but the most possible depth lies between 9 and 11 metres. Failure probabilities associated with these depths can be calculated; the failure probability will therefore lie between 0.006 and 0.082, but the most possible failure probability will lie between 0.011 and 0.032.

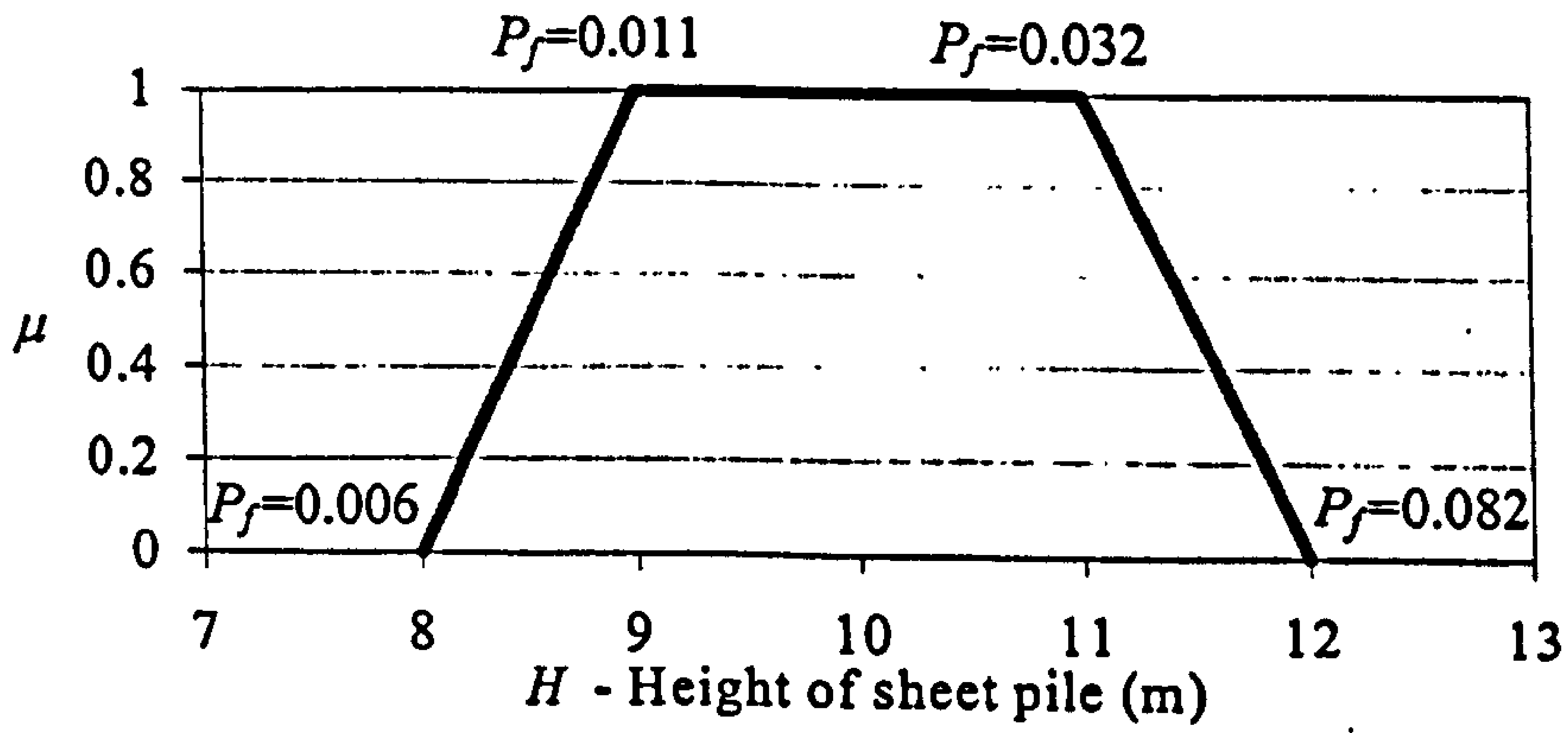


Figure 5.14 Fuzzy set defining sheet pile height and the corresponding failure probabilities

Whilst the above examples provide an indication of present performance, the decision-maker is often interested in how this performance is expected to change. If the beach is assumed to lower at a constant rate of 0.04m/year, the increase in annual failure probability can be predicted as shown in Figure 5.15.

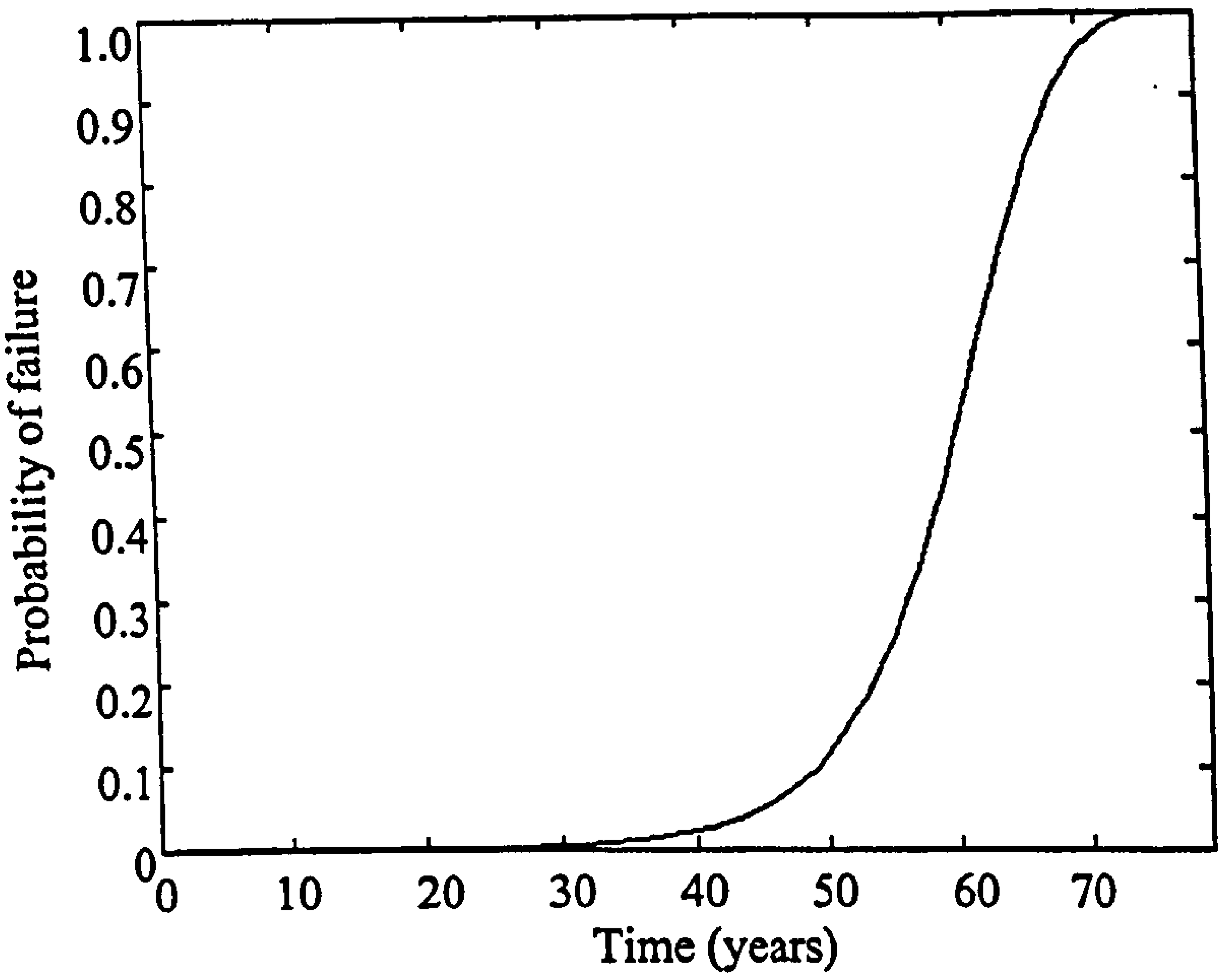


Figure 5.15 Change in annual rotational failure probability for a receding beach

The change in annual failure probability due to corrosion can be predicted by consideration of the change in moment capacity assuming a steady rate of corrosion of 0.035mm/side/year (Corus Group, 2001). Figure 5.16 shows the time dependent probability of failure due to moment capacity with a receding beach.

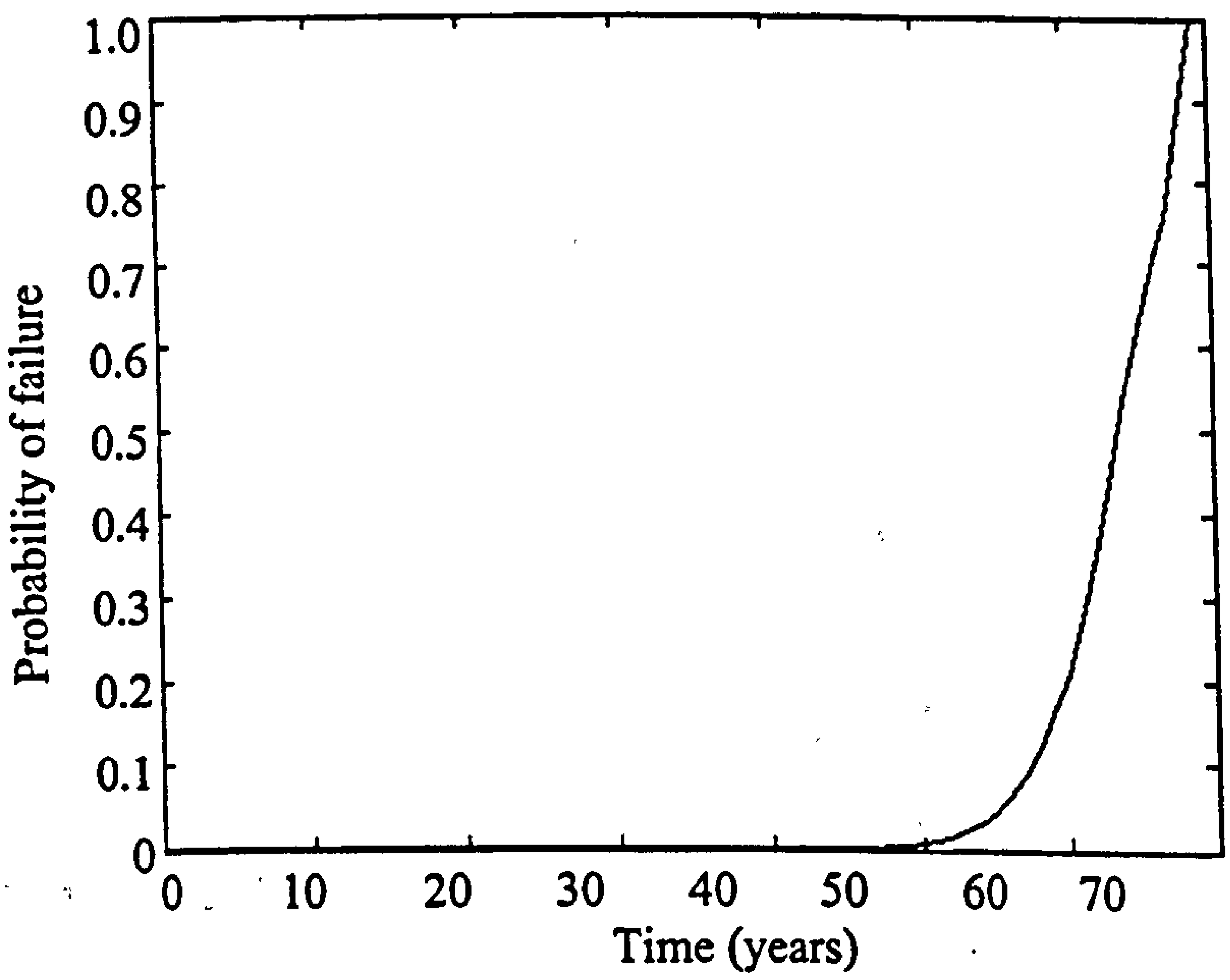


Figure 5.16 Change in annual corrosion failure probability over time for a sheet pile that is corroding on a receding beach

The first order series system failure bounds have been calculated using the equations defined in Chapter 3 (Section 3.3.3). The bounds represent the upper and lower limits of the failure probability assuming the failure modes are somewhere between completely dependent or independent.

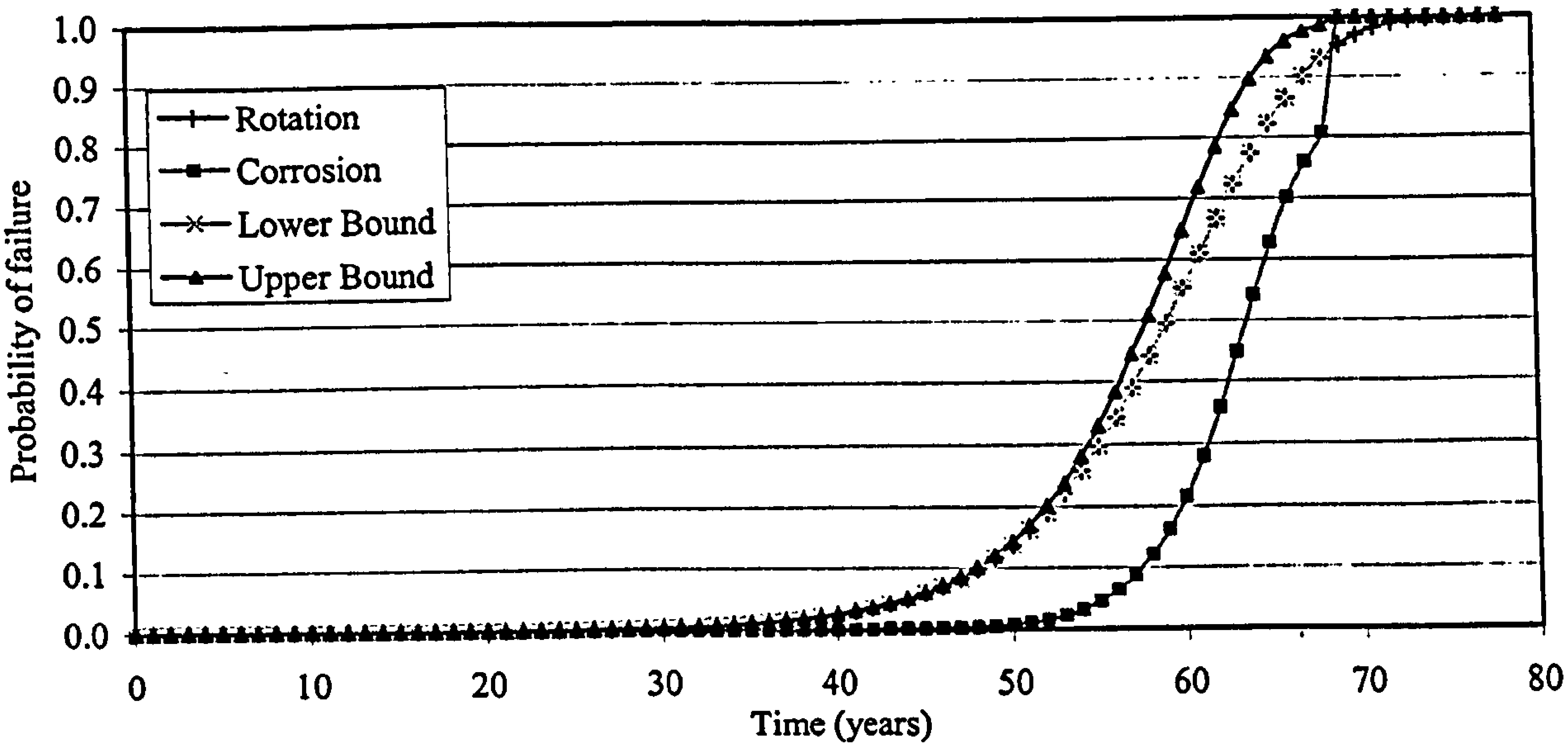


Figure 5.17 Upper and lower failure bounds for sheet pile retaining wall, considering both failure due to corrosion and toe erosion

The decision-maker can now monitor the bounds of system failure for multiple modes of structural failure over time as is shown in Figure 5.17. It is apparent that rotational failure due to insufficient toe depth dominates the failure bounds until year 70.

5.5. SUMMARY

This chapter responded to the need identified in Chapter 2 for developing a probabilistic approach to condition characterisation of flood defences. Reliability analysis has been shown to provide a useful approach to assessing the condition of flood and coastal defences. The approach to condition characterisation compliments the detailed level of the risk assessment methodology proposed in Chapter 4. As with the high level risk assessment, the condition of the structure is described using fragility curves which are a useful condition characterisation tool as they provide a representation of defence performance over a range of loading conditions. Integration of the fragility curve over the loading provides a failure probability of the defence. The natural uncertainties due to loading and epistemic uncertainties associated with the condition characterisation are kept separate.

Traditional reliability analysis only allows uncertainty to be expressed as probability distributions of random variables. This is often an inappropriate means of capturing the expert judgement so often associated with estimating some failure parameters. The reliability analysis has been adapted to allow membership functions to describe these parameters and it has been demonstrated using the example of a revetment how this uncertainty can be quantified and is still apparent in the final estimation of failure probability which is reflected as probability bounds or fuzzy probabilities. As a result of being able to quantify the uncertainty, the most useful monitoring information can be targeted to obtain the largest decrease possible at an appropriate price. This has been demonstrated by using uncertain information to characterise the condition of a revetment and a sheet pile.

Frequently, the decision-maker is interested in how a structure will perform over time. Models can be used to predict how a beach will recede or how a structure will corrode, and this information can be used to monitor how a structure's performance will very with time. This has been demonstrated by monitoring the change in the damage number in Van Der Meer's rock armour stability equation as a result of a time series of wave heights and also showing the loss in structural performance of a sheet pile due to corrosion and reduction of toe depth caused by beach lowering.

System bounds on failure probability can be estimated to provide an estimation of the defence's proneness to failure and consequently the proneness to failure of a defence system. System behaviour can be predicted and monitored over time providing a useful risk-based tool for the flood defence manager.

Chapter 6

Decision support for flood defence systems

6.1. OVERVIEW

In Chapter 2, it was shown that decision-making in flood defence is a multi-disciplinary endeavour involving a complex set of technical, economic, social and environmental issues. The processes of options analysis and evaluation in flood defence management involve assembling and manipulating vast quantities of evidence that can appear in a wide range of formats. Coupled with the sheer scale and complexity of the system there is potential for monitoring and remediation resources to be mis-directed.

In this chapter it is demonstrated how needs for improved decision-making may be addressed by the development of generic principles and a software tool that assists a non-expert user to implement those principles. An overview of the methodology summarises the key concepts and processes, which are then described in more detail. These concepts have been implemented in a case study of a flood defence system that demonstrates the applicability and use of the techniques for flood defence managers. The integration of the risk assessment and condition characterisation methodologies is also described, providing a useful approach to supporting flood defence managers.

6.2. KEY PRINCIPLES

The needs identified for decision-support in Chapter 2 have been satisfied by the development of generic principles and a software tool that assists a non-expert user to implement those principles. The main elements of the proposed modelling approach are illustrated in Figure 6.1. The photograph on the bottom left hand side of the diagram represents the flood defence system of interest. Abstracted from this are measurements of performance and a hierarchical model of the flood defence system. Performance indicators are associated with one or more relevant sub-systems in the hierarchical model.

A probabilistic condition characterisation provides evidence of the structural performance of a flood defence. This condition characterisation is also used as part of a risk assessment. Risk assessment provides an overall measure of the performance of the defence system (in terms of average annual risk) and also more detailed information on the contribution which individual defences make to flood risk. Whilst risk is an important indicator of the performance of flood defence systems, it is inevitably incomplete and will have to exclude information that is relevant to system performance because it appears in an inappropriate format or at an unsuitable scale. The decision-support methodology maximises the results of a risk assessment by incorporating them at the appropriate level of the system description, yet places risk in the broader context of all of the sources of evidence and values that the asset manager will wish to take into account.

Performance indicators are projected through value functions to provide a non-dimensional measure of how performance is valued in the context of a particular sub-system. Each indicator is then weighted to generate a figure of merit for that sub-system. A revised set of weightings are used to generate figures of merit for specific aspects of system behaviour. These concepts are discussed in more details in the following sections.

6.2.1. Process

A general terminology is required that can integrate both 'hard' (*eg.* flood defences) and 'soft' (*eg.* flood warning) aspects of the flood management system, focusing upon how they deliver performance (Hall and Davis, 2001). It is proposed that the concepts and terminology of process modelling can provide the required generic framework. A process is a purposeful activity, in that it enacts a transformation in a controlled manner. A typical process enacted by a flood dyke would be "Protecting town X from flooding". Note how the participle of the verb is often used in the description of a process. The process may be thought of as taking some resources, be they physical or information resources, and transforming them into an output. Figure 6.2 illustrates the generic process, with the process' primary function being the transformation it enacts within a control loop that ensures the process continues to deliver the required performance in a dynamic environment. In the context of flood defence this usually involves investment of capital or operational budget which results in some transformation in terms of risk reduction. The transformation is enacted by a sub-system (for example a flood dyke) within the overall system under consideration, providing a link between the process that delivers a desired response and the sub-system that enacts that process. Using processes rather than physical entities in the framework allows a much richer description of the performance of the sub-system which may include a range of project management issues as attributes of the processes.

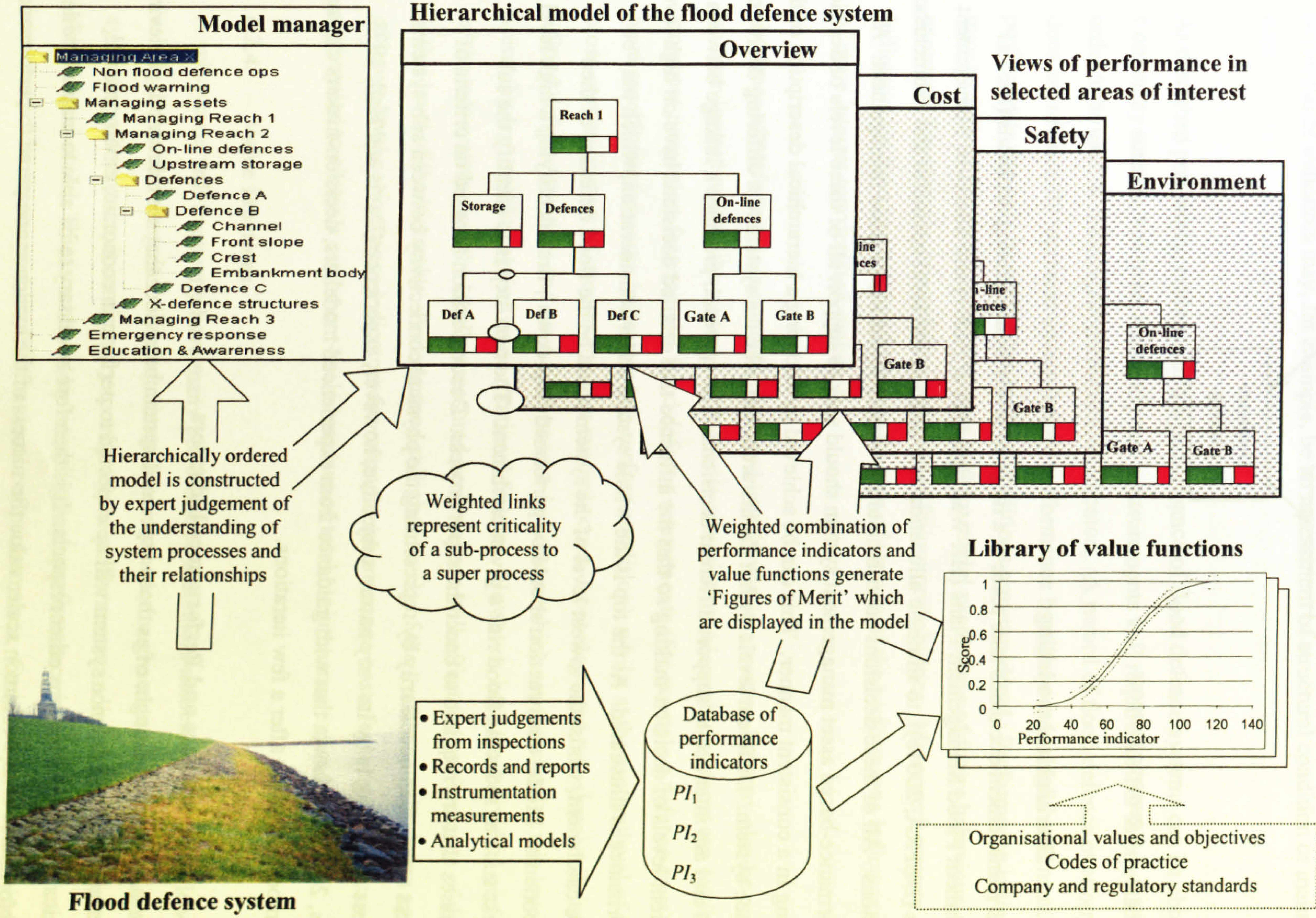


Figure 6.1 An overview of the performance-based management methodology schematics

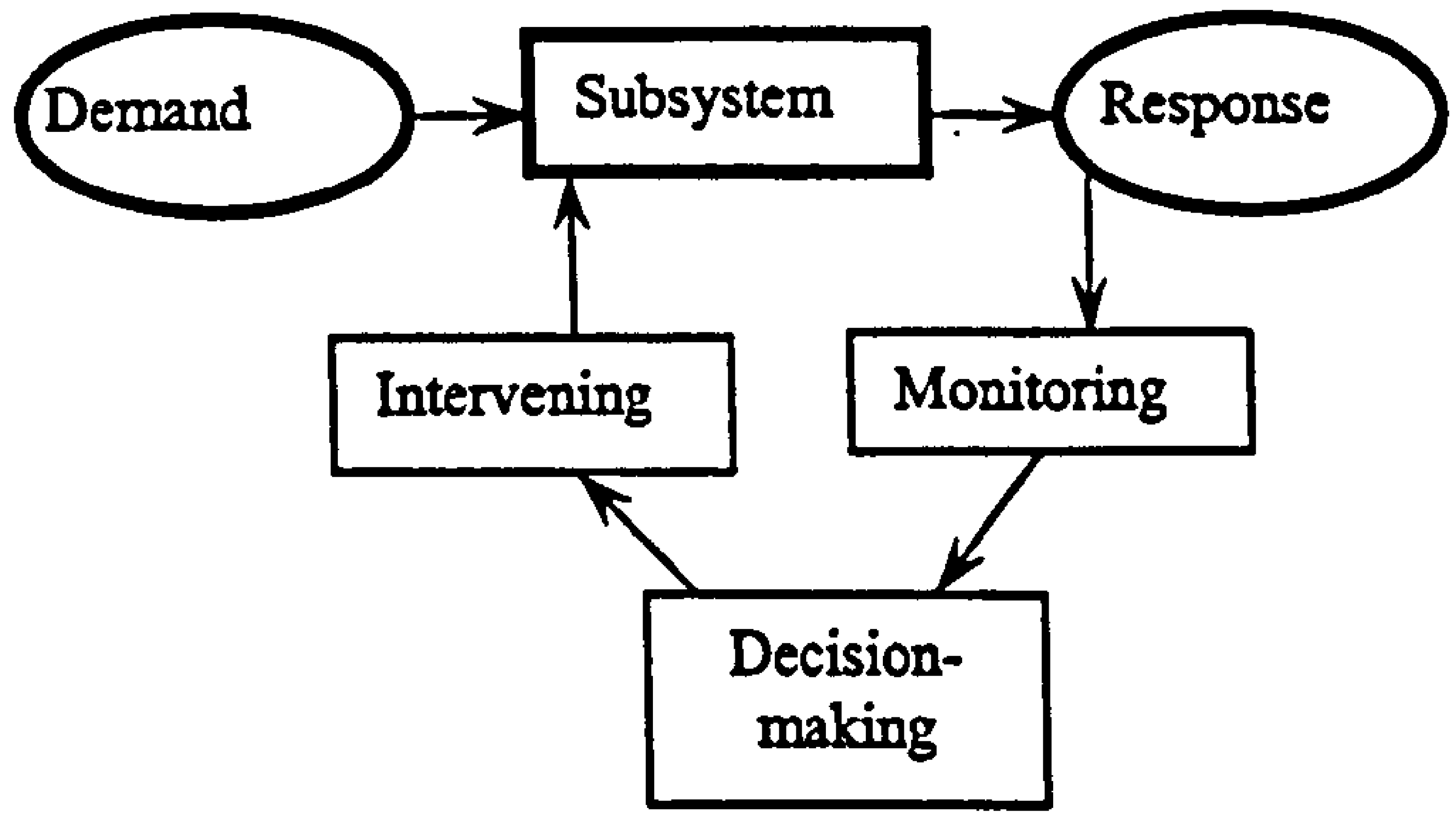


Figure 6.2 A generic process

6.2.2. Hierarchical modelling

Flood management decisions take place at a range of levels from planning decisions (such as Catchment Flood Management Plans (HR Wallingford *et al.*, 2001) and Shoreline Management Plans (DEFRA, 2001c)), to strategic allocation of resources, and local operation and maintenance decisions. All of these decisions contribute to the overall objectives of flood management. A performance-based asset management system should seek to support all of these levels of decision-making in a consistent manner. This can be achieved by adopting a hierarchical description of the defence system. Different levels within the hierarchy describe the system to a differing degree of detail and are intended to support different decisions. The approach provides a linkage between different levels of decision-making so that the influence that detailed decisions have on high level performance is illustrated. At the top is the whole system under consideration, which may be a whole catchment, whilst the lowest level of the system could be the flood defences or their components. Model construction should be performed 'top-down', first identifying a high level process such as reducing flood risk a given catchment. The next step is to identify the sub-processes that are required to enact the super process. Decomposition is based on criteria of process rather than necessarily by representing the physical connectivity between sub-systems. Process modelling is by its very nature subjective, however, experience (Davis and Hall, 1998, Davis, 2002) has shown that with guidance from experienced modellers, decision-makers can agree on a model structure after a few iterations.

6.2.3. Performance and Performance Indicators

Performance can be thought of as those aspects of system behaviour that are relevant to achieving objectives. A flood defence system will be expected to perform in economic, environmental, technical, safety and perhaps other respects. It is therefore necessary to be able to map all evidence of performance onto a common scale, value the success of the evidence compared to performance objectives and weigh the relative performance of different pieces of evidence against each other.

Evidence about performance of a system or sub-system is provided by performance indicators. These performance indicators are the system state variables that are relevant to achieving objectives. System objectives are derived from organisational and stakeholder values. In flood management, high level performance indicators are derived from government policy and include, for example, total national flood risk (reported on by the National Flood Risk Assessment 2002 (HR Wallingford *et al.*, 2003) study) or annual investment in flood defence. Low level performance indicators may, for example, be an assessment of structural condition or maintenance cost.

As described previously, evidence of the performance of flood defence systems can be obtained from many sources, however, frequently raw measurements will require further interpretation in order to be used as meaningful performance indicators. A record of flood defence overtopping does not necessarily represent unsatisfactory performance because a meaningful statement of performance can only be made after loading conditions are considered. Performance indicators may be derived from monitoring activities or from model predictions of future behaviour, expressed for example as inundation contours or failure probabilities. Many performance indicators are time varying, in which case the rate of change of the performance indicator may be of more interest to a decision-maker.

Evidence of performance is not only measured locally within a system, but may also be propagated up from lower levels of the system. At higher levels within a hierarchy, systems can exhibit emergent properties, for example, flood defence components (for example a defence crest) will have specific performance indicators (*eg.* surface condition), whilst at the system level of the dyke cross-section, groups of components (for example crest, front slope and rear slope) will interact as a flood defence structure and have performance indicators which are measured at this level of the hierarchy (*eg.* stability against mass rotational failure). However, both the individual performance of the sub-systems and the locally measured performance will affect the performance of the system at this level of the hierarchy. Performance at higher levels of the system can also be an aggregation of lower level performance indicators for example maintenance costs for a river reach can be aggregated to obtain a catchment-scale measure of total maintenance expenditure.

6.2.4. Value functions

A sub-system will frequently have many performance indicators associated with it, which will usually be measured against different dimensions. In order to generate an overview of the sub-system's performance, it is necessary to map all the performance indicators onto a common non-dimensional scale. This is achieved by mapping each performance indicator with a function that represents how the user values different levels of performance. Once the performance indicators have been mapped onto a non-dimensional scale, they are weighted to reflect their relative importance to achieving the sub-system's objective.

Figure 6.3 shows the general shape of these value functions. The value of v_i, the measure of how the sub-system, i, is performing is generated by projecting the performance indicator, PI_i, through the value function f_i:

$$v_i = f_i(PI_i) \tag{6.1}$$

For example, using a linear value function (Figure 6.3(a)) to value a performance indicator measured as being 63, on a scale of 0 (worst performance) to 100 (maximum performance) results in a value of 0.63. The value function is chosen by the user based on organisational values and objectives, codes of practice and company and legal standards. A stepped function (shown in Figure 6.3(b)), for example, could be used to represent a regulatory threshold, where performance one side of the step would be 'acceptable' and given a score of 1, conversely performance the other side is 'unacceptable' and given a score of 0.

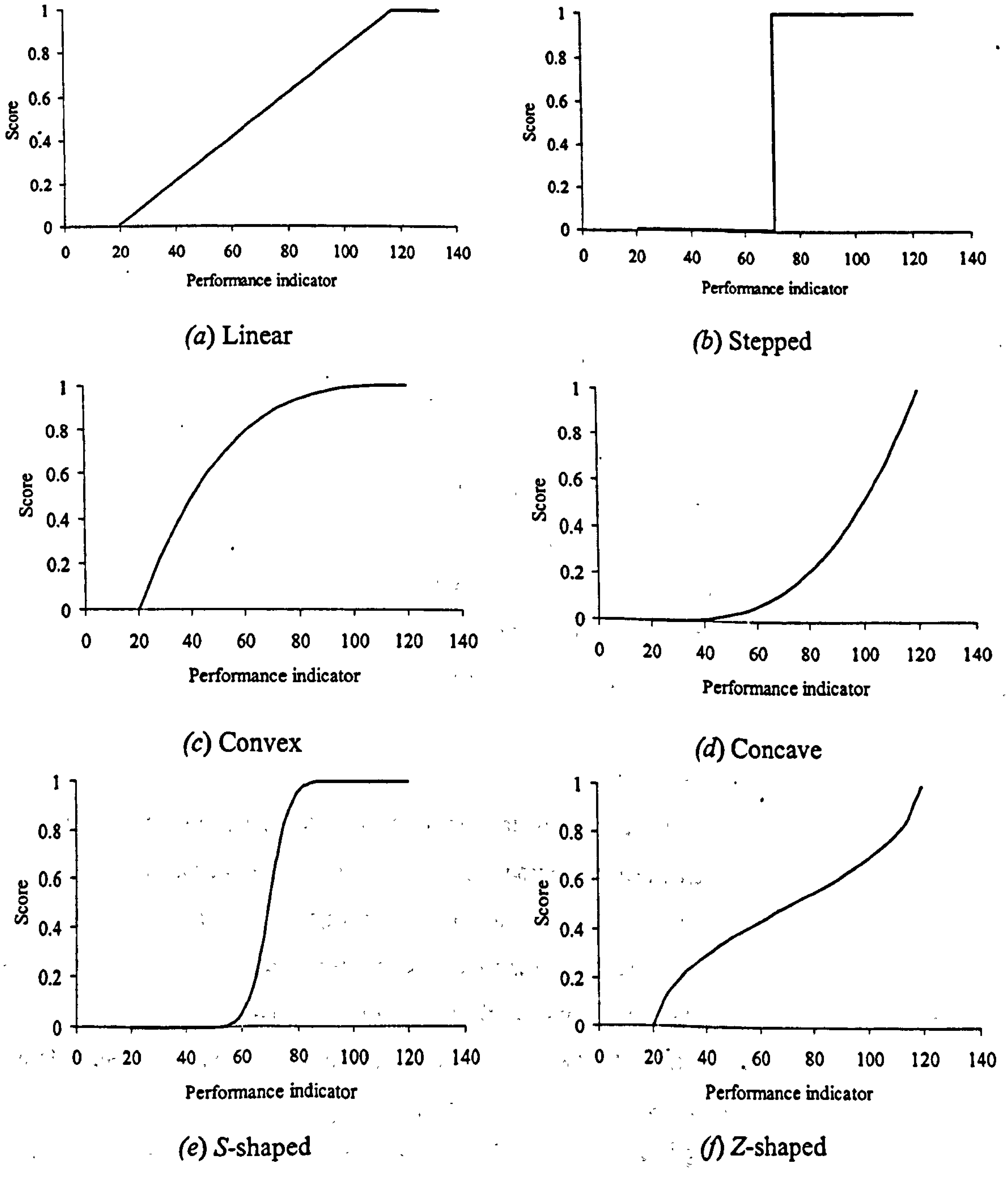


Figure 6.3 Value functions used to map performance onto a non-dimensional scale

Under some circumstance a performance indicator may be projected through more than one value function in order to reflect how a particular aspect of performance is evaluated against multiple objectives. For example the indicator "frequency of defence inspection" may be evaluated with respect to system reliability (in which case higher frequency represents improved performance) and also with respect to cost (in which case higher frequency represents lower performance).

Figure 6.4 demonstrates how a performance indicator, in this case crest height, is mapped through a value function on to a figure of merit (Section 6.2.5) represented by an 'Italian flag' motif, which is a graphical representation of an interval probability where green (the lighter shade on the left hand side of the tricolore) represents evidence of satisfactory performance, S_n, conversely the red (the darker shade on the right hand side of the tricolore) represents unsatisfactory performance defined by $1\text{-}S_p$ and white represents the uncertainty or $S_p\text{-}S_n$. Figure 6.4 shows a measurement of crest height of 45m AOD measured to an accuracy of ±2cm that is mapped through an *s*-shaped value function which also has a degree of uncertainty associated with it. This represents the uncertainty with the river routing models that predicted the flood event that would overtop the defence and any lack of confidence with the choice of value function (perhaps if a group of experts were unable to agree on a particular value function). The values of S_n and S_p are 0.1 and 0.87 respectively.

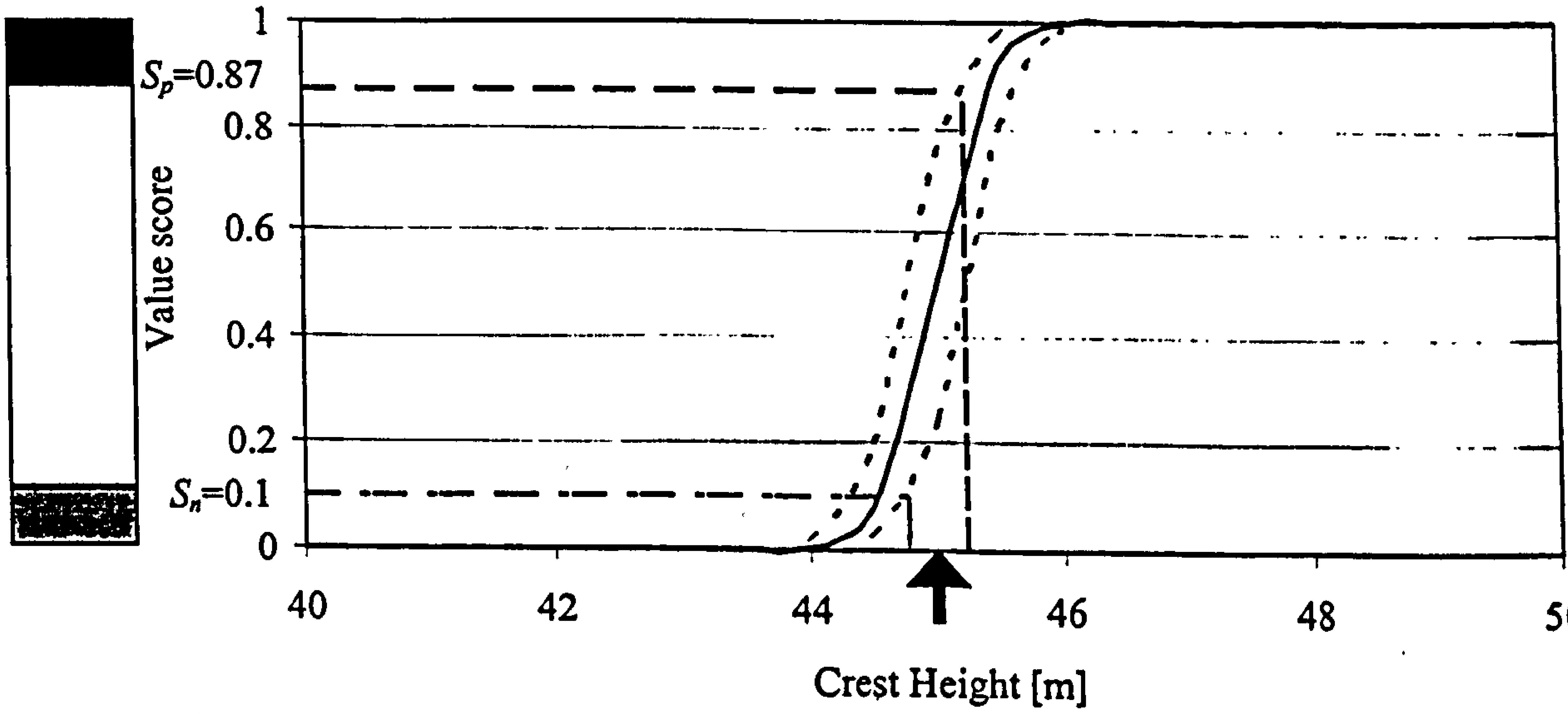


Figure 6.4 Mapping a measurement through a value function to generate a non-dimensional value score

A special case of uncertainty is when performance indictors are recorded as linguistic values, for example on a five-word scale from 'very poor' to 'very good' such as that currently in use by the Environment Agency to assess the condition of flood defences. This measure defines the level of performance and whilst the uncertainty associated with this is captured in terms of the vagueness of the measurement, the degree of vagueness will depend on the performance indicator being measured and the experience and knowledge of the engineer making the measurement. Assigning an assessment of confidence that is also on a five point scale allows these factors to be captured and also makes it is possible to generate an interval measure as shown in Figure 6.5. For example a judgement of 'good' performance made with 'medium' confidence yields an interval probability of [0.62, 0.87]. Linguistic performance indicators are elicited directly from the expert as a judgement of performance relative to objectives so there is no need to project the performance indicator through a value function.

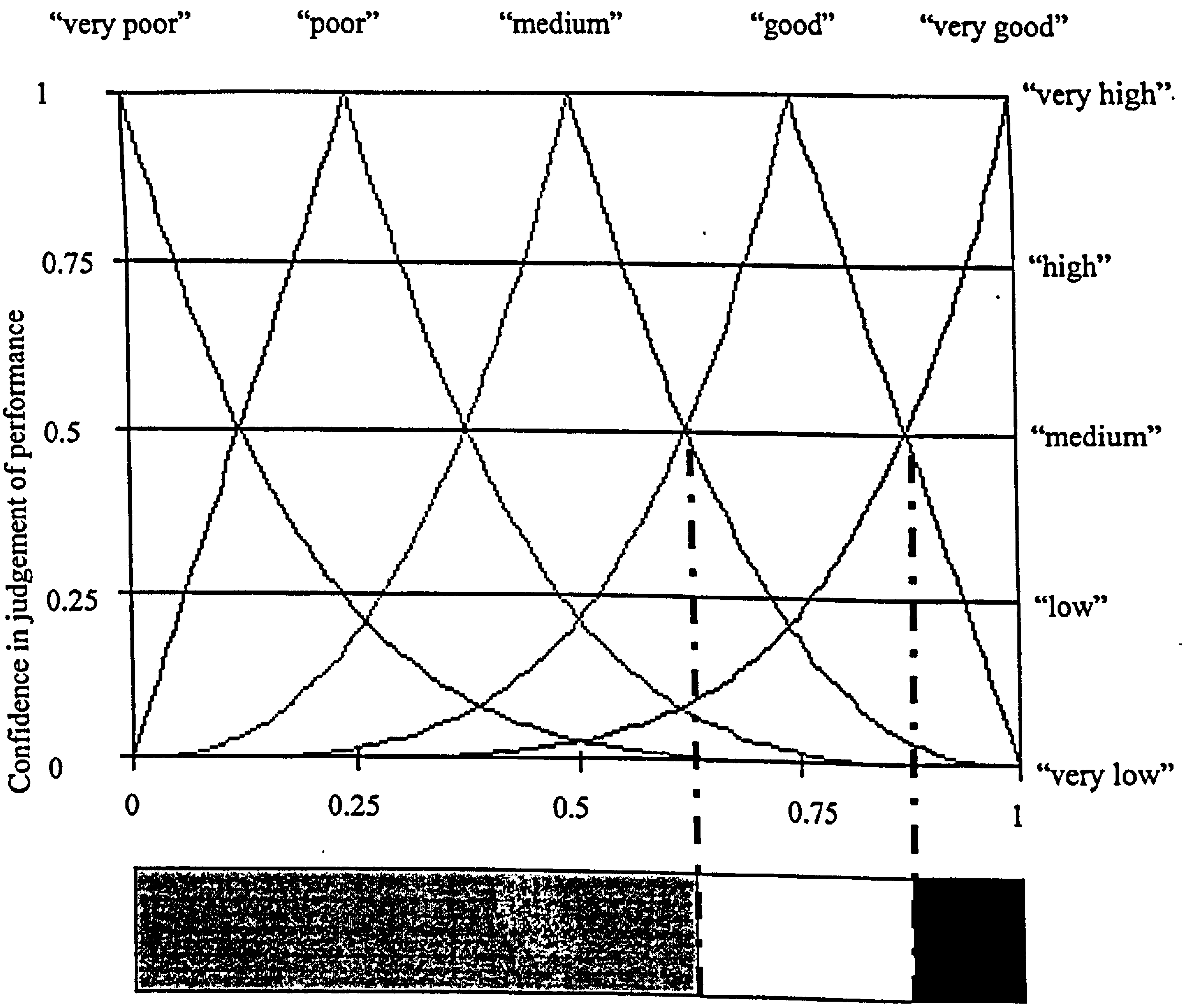


Figure 6.5 Mapping linguistic evidence onto a non-dimensional scale using value functions

6.2.5. Figure of Merit

Decision-makers are interested in how a system is performing against a range of objectives (Ozelkan and Duckstein, 1996). This overview of system performance, taking into account all objectives (*eg.* cost, environment, safety) is called a figure of merit. For sub-system *i* with *m* performance indicators, the figure of merit, FM_i, is calculated by applying a weight, *w*, which represents the importance of the performance indicator to the overall performance of the system (Wymore, 1993):

$$FM_i = \sum_{j=1}^{m} w_{i,j} v_{i,j}\,, \quad \sum_{j=1}^{m} w_{i,j} = 1 \tag{6.2}$$

where the weights $w_{i,j}$ represent the relative importance of the value, $v_{i,j}$, of the performance indicators to the sub-system's overall performance.

Decision-makers are also interested in performance of the system against specific attributes such as cost or safety. These can be thought of as different aspects of performance, the new figure of merit, $FM_{i,A}$, representing performance against aspect, A, is calculated by altering the weightings according to the aspect of system performance under consideration as shown in Equation 6.3.

$$FM_{i,A} = \sum_{j=1}^{m} w_{i,j,A} v_{i,j}, \qquad \sum_{j=1}^{m} w_{i,j,A} = 1 \tag{6.3}$$

Different aspect views are represented by the multiple sheets in the top right corner of Figure 6.1.

6.3. PROPAGATING EVIDENCE USING INTERVAL PROBABILITY

Performance indicators and figures of merit will inevitably have uncertainty associated with them. This is due to both errors in measurements and predictions, and uncertainties in how performance is valued and compared. A simple approach to uncertainty handling has been adopted using interval bounds for all uncertain quantities, including measurements and value functions (see Figure 6.4). The following sections describe how, using interval probability theory, evidence of performance is propagated through the hierarchical model of the system.

6.3.1. Propagating evidence through a hierarchy

Figure 6.6 provides an overview of how evidence from performance is propagated through the system.

Evidence of performance can be measured at a specific level in the hierarchy and also be propagated up the hierarchy from lower sub-systems. A figure of merit for the propagated evidence and figure of merit for the direct evidence (calculated using Equation 6.2) are weighted to provide a merged figure of merit, FM_{merged}, for a given sub-system such that:

$$P(FM_i)_{merged} = w_1.P(FM_i)_{measured} + w_2.P(FM_i)_{propagated} \quad , \quad w_1 + w_2 = 1 \tag{6.4}$$

Calculation of the performance of the super-system requires consideration of how the sub-systems and performance evidence interact, and this is represented by necessity, sufficiency and dependency. The following sections describe these measures and how, through use of Interval Probability Theory, evidence is propagated through the hierarchy.

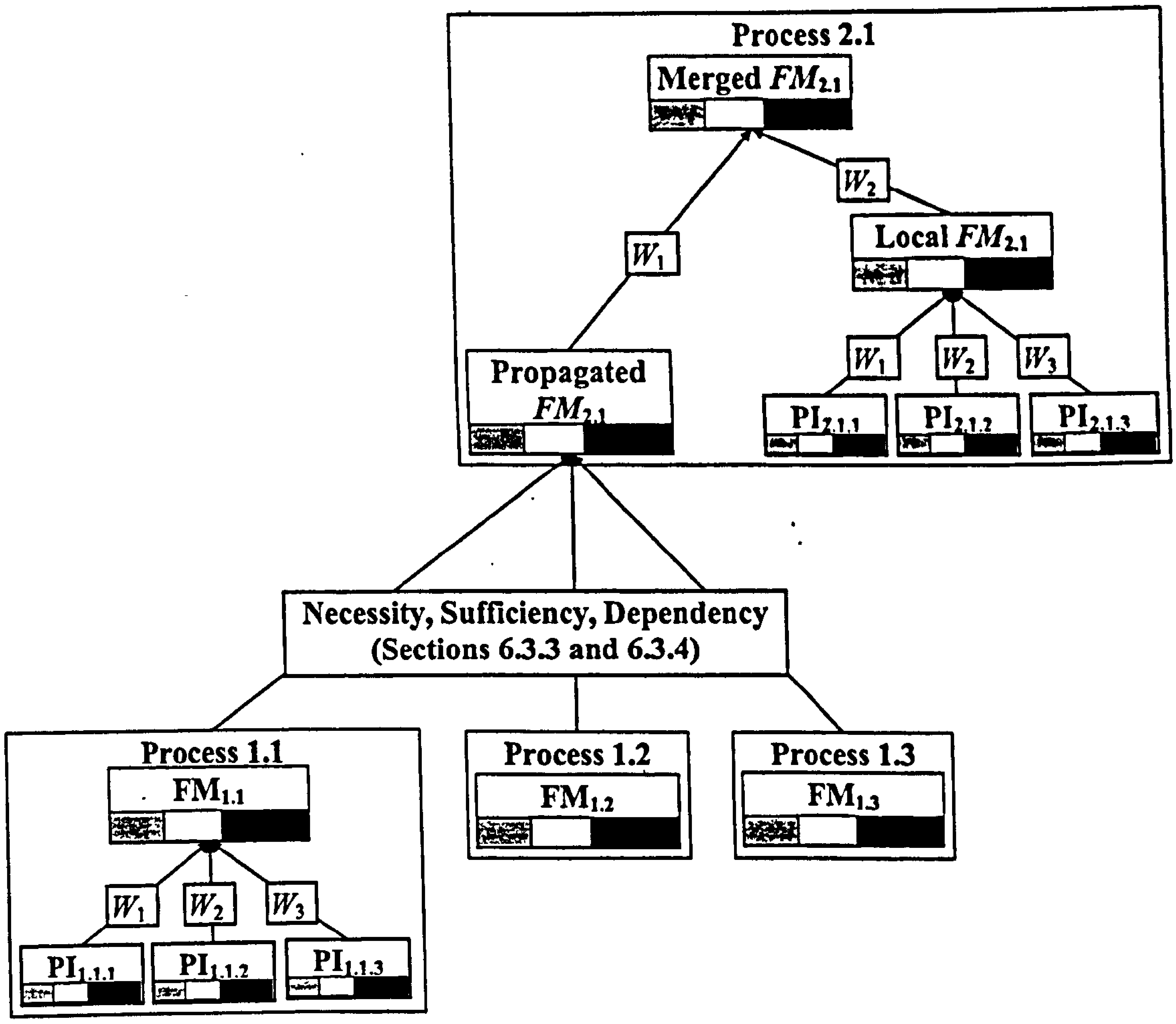


Figure 6.6 Overview of evidence propagation

6.3.2. Interval probability theory

Evidence is propagated through the hierarchy using interval probability theory (Cui and Blockley, 1990, Hall *et al.*, 1998) to represent the degree of belief that a process is performing well when compared with objectives. If E represents the proposition about the satisfactory performance of a system, the probability $P(E)$ is defined by a lower bound, $S_n(E)$, and an upper bound, $S_p(E)$.

$$P(E) \in [S_n(E), S_p(E)] \tag{6.5}$$

The interval probability is interpreted as a measure of belief, where $S_n(E)$ represents the extent to which it is certainly believed that E is true or dependable, $1 - S_p(E) = S_n(\bar{E})$ represents the extent to which it is certainly believed that E is false or not dependable, and the value $S_p(E) - S_n(E)$ represents the extent of uncertainty of belief in the truth or dependability of E. Three extreme cases illustrate the meaning of this interval measure of belief:

- $P(E) \in [0,0]$ represents a belief that E is certainly false or not dependable, meaning there is no satisfactory performance,
- $P(E) \in [1,1]$ represents a belief that E is certainly true or dependable , meaning there is no unsatisfactory performance, and,
- $P(E) \in [0,1]$ represents a belief that the degree of performance is unknown.

The interval $S_n(E)=S_p(E)$ implies that there is no uncertainty in the proposition and corresponds to conventional probability. This approach is founded in evidence theory (Shafer, 1976) which can be used where handling uncertainty with probability theory is badly suited due to vagueness or incompleteness in information (Klir and Folger, 1988). Interval probability theory (IPT) is useful as it retains the desirable properties that make evidence theory more attractive than Bayesian probability methods (Hall *et al.*, 2002b) as it:

- represents in a fairly straightforward manner aspects of ambiguity, vagueness and incompleteness in evidence,
- provides a balance between, on the one hand, not being so weak as to provide inferences that are of limited practical use, yet on the other hand not artificially constraining the problem implying less uncertainty than is in fact the case,
- conveniently represents dependency relationships between evidence which is an important issue in complex evidential situations,
- captures a range of inferential relationships between levels in the evidence hierarchy, and,
- is closely related to probability theory, however, it is not necessary to exclusively allocate probability to a judgement in line with belief and plausibility measures (Shafer 1976) and possibility measures (Zadeh, 1978) that have been introduced in Chapter 3.

6.3.3. Dependency

The dependency between sub-systems represents the amount of evidence originating from a common source or being influenced by common processes. For example there would be a high dependency between the age of an asset and its condition recorded upon inspection. Both pieces of evidence could be used in an assessment of performance, but if they were combined without recognising the dependency then the overall evidence for satisfactory performance may be overestimated.

Cui and Blockley (1990) developed previous work on interval representations by introducing the parameter, ρ, which represents the degree of dependence between propositions E_1 and E_2:

$$\rho = \frac{P(E_1 \cap E_2)}{\min(P(E_1), P(E_2))}. \tag{6.6}$$

Thus $\rho = 1$ indicates that $E_1 \subset E_2$ or $E_2 \subset E_1$ (*i.e.* they are nested propositions), whilst if E_1 and E_2 are independent

$$\rho = \max(P(E_1), P(E_2)) \tag{6.7}$$

so that

$$P(E_1 \cap E_2) = P(E_1).P(E_2). \tag{6.8}$$

The minimum value of ρ is given by

$$\rho = \max\left[\frac{P(E_1) + P(E_2) - 1}{\min(P(E_1), P(E_2))}, 0\right] \tag{6.9}$$

where $\rho = 0$ indicates that E_1 and E_2 are disjoint or mutually exclusive such that $E_1.E_2=\emptyset$.

If ρ is defined as an interval $[\rho_l, \rho_u]$ then:

$$S_n(E_1 \cap E_2) = \rho_l.\min(S_n(E_1), S_n(E_2)) \tag{6.10}$$

$$S_p(E_1 \cap E_2) = \rho_u.\min(S_p(E_1), S_p(E_2)) \tag{6.11}$$

$$S_n(E_1 \cup E_2) = S_n(E_1) + S_n(E_2) - \rho_l.\min(S_n(E_1), S_n(E_2)) \tag{6.12}$$

$$S_p(E_1 \cup E_2) = S_p(E_1) + S_p(E_2) - \rho_u.\min(S_p(E_1), S_p(E_2)). \tag{6.13}$$

To implement the methodology, a dependency parameter is assigned on a scale [-1, 1] with 0 representing independence. A linear transformation converts this to the actual dependency, ρ, on a scale [0, 1]. The approach has been adopted because, from the definition of ρ in Equation 6.9, it can be seen that the value of ρ representing independence varies with the level of evidence assigned to or calculated for the relevant sub-processes. The use of the linear transformation means that the user can make a simple judgement of dependency on a three point scale from mutual exclusion (-1) to independence (0) to full dependence (1) without having to calculate ρ. This relationship is given in Table 6.1, points between the three defined values are extrapolated linearly.

Table 6.1 The relationship between the dependency parameter and ρ for two propositions

	Mutually exclusive	Independent	Dependent
ρ	0	$\max(P(E_1), P(E_2))$	1
Dependency parameter	-1	0	1

The dependency parameter is an additional item of information, which is elicited in order to address explicitly the dependency between propositions. It is a convenient means of exploring different dependence relationships when the exact nature of dependence is uncertain. A measure of dependency is defined between each sub-process. An example calculation showing the use of dependency in interval probability theory is given in Appendix H.

6.3.4. Logical Inference, Necessity and Sufficiency

Having established a method for combining probabilities, the relationship between the propositions, $E_1, E_2 \ldots E_n$ about the performance of sub-systems $1,2\ldots n$ and some hypothesis, H, about the performance of their super-system needs to be addressed. To establish the support, $P(H)$, on the basis of the propositions, it is necessary to know $P(E)$ and the relationship between E and H. This relationship is defined by the conditional measures $P(H|E)$ and $P(H \mid \overline{E})$ which can be obtained using the theorem of total probability (Chapter 3). For a single proposition this is:

$$P(H) = P(H \mid E).P(E) + P(H \mid \overline{E}).P(\overline{E}) \tag{6.14}$$

which can be rewritten as:

$$P(H) = P(H \mid E).P(E) + P(H \mid \overline{E}).(1 - P(E)) \tag{6.15}$$

This has been addressed for one proposition by Dubois and Prade (1990), when all the terms are expressed as interval number the bounds S_n and S_p are:

$$S_n(H) = S_n(H \mid E)S_n(E) + S_n(H \mid \overline{E})(1 - S_n(E)) \;;\; S_n(H \mid E) \geq S_n(H \mid \overline{E}),$$

$$S_n(H) = S_n(H \mid E)S_p(E) + S_n(H \mid \overline{E})(1 - S_p(E)) \;;\text{ otherwise} \tag{6.16}$$

and

$$S_p(H) = S_p(H \mid E)S_p(E) + S_p(H \mid \overline{E})(1 - S_p(E)) \;;\; S_p(H \mid E) \geq S_p(H \mid \overline{E}),$$

$$S_p(H) = S_p(H \mid E)S_n(E) + S_p(H \mid \overline{E})(1 - S_n(E)) \;;\text{ otherwise} \tag{6.17}$$

If E is a *necessary* condition for H (Figure 6.7(a)):

$$P(H \mid E) \leq 1, P(H \mid \overline{E}) = 0 \tag{6.18}$$

If E is a *sufficient* condition for H (Figure 6.7(b)):

$$P(H \mid E) = 1, P(H \mid \overline{E}) \leq 1 \tag{6.19}$$

In the special case that E is both a *necessary* and *sufficient* condition for H:

$$P(H \mid E) = 1, P(H \mid \overline{E}) = 0 \tag{6.20}$$

A weaker and more general condition is when E is *relevant* or *partially sufficient* to H (Figure 6.7(c)), in which case:

$$0 < P(H \mid E) \leq 1, 0 \leq P(H \mid \overline{E}) \leq 1 \tag{6.21}$$

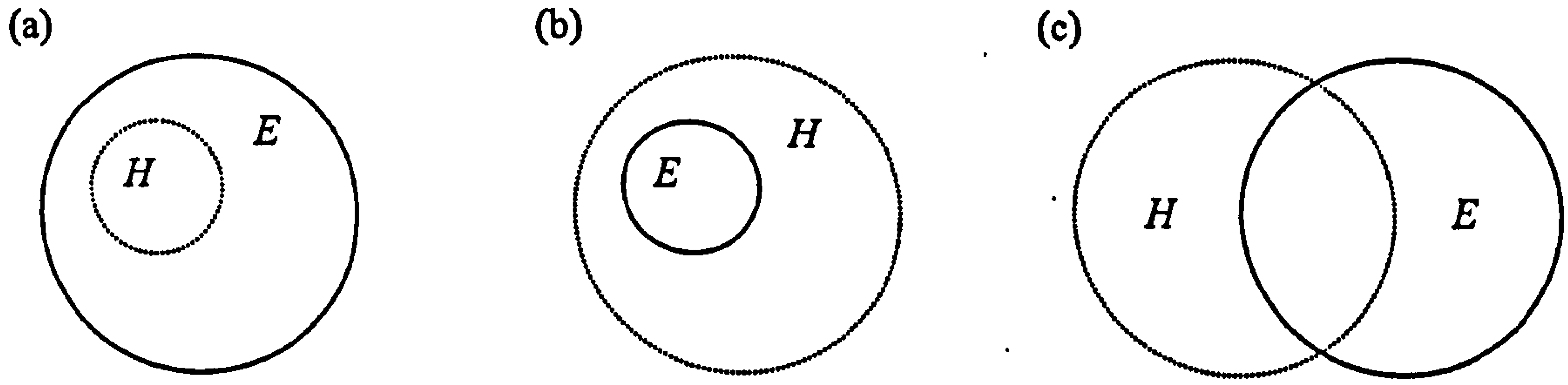


Figure 6.7 Venn diagrams of (a) E necessary for H; (b) E sufficient for H; (c) E relevant to H

In the context of performance modelling of systems, the *sufficiency*, S, is a measure of the influence that a given sub-system has on the performance of its parent or super-system, and, the *necessity*, N, is a measure of the extent to which failure (non-performance) of a sub-system will cause failure (non-performance) of its parent super-system. Increasing sufficiency has the effect of increasing the weight applied to the given sub-system. Increasing necessity has the effect of decreasing the weight applied to any sub-systems that do not intersect (are not dependent upon) the given sub-system. If a sub-system is necessary to avoid failure, it provides importance evidence about the super-system so the weight applied to those sub-systems that do not intersect with it (*i.e.* intersect with its negation) should be small. For example, a flood warning system that performs well will greatly improve the overall performance of the flood defence system meaning it will have a high sufficiency. However failure of the flood warning system does not result in complete failure of the flood system as other system processes are still functioning, therefore the necessity will be quite low.

This methodology has been developed further by Hall *et al.* (1998b) to generate bounds on the inference for systems with *n* proposition. Due to the lengthy nature of this calculation, a worked through example is provided in Appendix H.

6.4. SOFTWARE IMPLEMENTATION *

The methodology outlined above is rather laborious to implement in practice without the help of a software tool for constructing process models, storing performance indicators and propagating evidence. A Windows based software tool (known as PERIMETA) has therefore been developed by co-researchers to support the methodology and demonstrate its applicability with a case study. The tool is described briefly here before describing in more detail the case study that formed part of the current research. The tool comprises of a hierarchical systems model linked to a database of performance indicators, with the following key elements:

- a graphical tool for drawing hierarchical models,
- a model manager, to navigate large models and switch between alternative special views,
- a database of performance indicators, which is intended to be compatible with an organisation's database and intranet systems,
- a graphing tool for illustrating how performance indicators have varied with time,
- a library of parameterised value functions, which can be chosen and adapted by the user, and,
- an inference engine for implementing IPT.

The software's help documents have been placed in Appendix I to provide a more detailed overview of how the software is used.

6.4.1. Graphical model construction

Each process model is constructed using a conventional drag and drop interface. Only acyclic (hierarchical) structures are permitted by the graphing tool. The model can be automatically sorted into layers.

The Figure of Merit provides an immediate overview of system performance, enabling the user to identify areas of poor performance and their implications. By clicking on the icon for a sub-system, the user is able to view:

- the interval estimates for measured and propagated evidence,
- the weights used to merge the measured and propagated evidence, and,

* The software developed for implementing the decision-support methodology was programmed by co-researchers, with design input from the author, however a brief overview is included here and more detailed information is provided in Appendix I.

- the necessity and sufficiency and dependency measures used in the propagation process.

Further interrogation takes the user to the relevant entries in the database of performance indicators (Section 6.4.3) and the value functions through which they have been projected.

6.4.2. Model manager

The hierarchical model can rapidly grow to be too large to show on a single sheet. Therefore, a 'model manager' is illustrated in the top left hand corner of Figure 6.1, which allows the user to navigate the model and show the area of interest. The user can collapse or expand sub-systems at the bottom of the model. The default view of the model displays the general values of the figures of merit. However, the user can switch between views by choosing a specific view from a tool bar (Section 6.2.5).

6.4.3. Database of performance indicators

Every performance indicator is held in a database which records:

- the name of the performance indicator,
- its current value,
- its dimensions,
- the sub-system(s) it provides information about, and,
- a default value function.

6.4.4. Library of value functions

When associating a performance indicator with a sub-system, the user has to specify the value function. This is achieved by choosing either the appropriate linguistic measure (Figure 6.5) or one of the six generic shapes shown in Figure 6.3 and specifying its limits and, if appropriate, its curvature. Uncertainty in the value function is handled by specifying an interval range within which the function can vary. Since, in general, both the performance indicator and the value function will be an interval-value, interval bounds on Equation 6.2 are calculated.

6.4.5. IPT inference engine

Having established the model structure and captured the performance indicators, the final step is to enable propagation of evidence through the hierarchy. At each level in the hierarchy other than the lowest level, the user enters

- 'necessity' and 'sufficiency' values, which represent the criticality of the performance of the sub-systems to the performance of their super-system, and,
- a 'dependency' value representing the strength of dependency between each of the sub-systems.

These are then automatically combined with the figures of merit of the sub-systems to generate the propagated estimate $P(H)_{propagated}$ of the figure of merit. Weights are entered to enable the merger of $P(H)_{measured}$ and $P(H)_{propagated}$ to calculate the value of $P(H)_{merged}$ for display in the graphical model and propagation up the hierarchy.

6.5. CASE STUDY: BURTON-UPON-TRENT

The case study focuses on a flood defence system in the Midlands, England that was modelled to demonstrate how the methodology can be used for a complex system and provide a useful overview of performance. The system comprised of the town, Burton-upon-Trent, which is situated on the river Trent and has approximately 10km of flood defence protecting more than 13000 people and 6000 properties (Figure 6.9). The higher level processes of the finished model are shown in Figure 6.14. Figure 6.15(a) and (b) show the lower level process which correspond to the physical defence assets.

The model has been populated with all the available evidence of the physical defence system and has therefore been decomposed to the lowest level at which this evidence is recorded. Where possible the other process in the system, such as for the flood warning branch of the hierarchy, have been populated with EA data. Where performance indicator values could not be obtained, for example, in the case of the planning control processes, values have been fabricated to demonstrate how these soft aspects of the flood defence system can be incorporated into the overall system model.

6.5.1. Model construction

The nature of flood defence decision-making means it is highly amenable to hierarchical modelling. The management of flood defence in England and Wales is hierarchical, with policies and high level targets defined at a national level. The next level of management is to identify long term plans at catchment and coastal cell level. Finally, at a more local level, strategies and then solutions are identified to realise these plans and implement them. Decisions on key tasks such as flood warning, planning control and operations and maintenance are made at all levels of this hierarchy (Harman *et al.*, 2002).

The focus of the case study was on the lower level systems that are dominated by the flood defence assets. Both the 'hard' system processes such as 'managing flood defences' and 'soft' systems such as 'operating a flood warning system' were integrated into the model. However, case studies in the dam sector have demonstrated that the methodology is robust enough to handle processes at more abstract levels of the infrastructure system (Hall *et al.*, 2002b). The nature of flood defence management means that there are a number of generic model levels that are common to a typical flood defence system, these are shown in Figure 6.8. This is clearly reflected in the structure of the full case study process model in Figure 6.14 and Figure 6.15.

Construction of the model was an iterative process involving a number of experts in the fields of flood and coastal defence. The top level of interest is the performance of the entire flood defence system. This is achieved by 'managing and providing defence infrastructure', 'operating a flood warning system', 'co-ordinating an emergency response', 'increasing public awareness' and 'controlling planning and advising planning bodies'. These processes can be further decomposed, for example, 'operating a flood warning system' is achieved by 'gathering data on rainfall and river flows', 'predicting flooding' and 'disseminating warning'. 'Managing defence assets' is decomposed into what appears to be a hierarchy of assets. Whilst the structure of the defence system is naturally hierarchical, it should not be forgotten that each 'box' still represents a dynamic process, not just an inanimate object. For example 'Embankment' represents not only the structure, but the control systems and those responsible for operating it. The model has been decomposed to the lowest level at which performance information is ordinarily acquired, which is the level of flood defence structural components, for example defence crest.

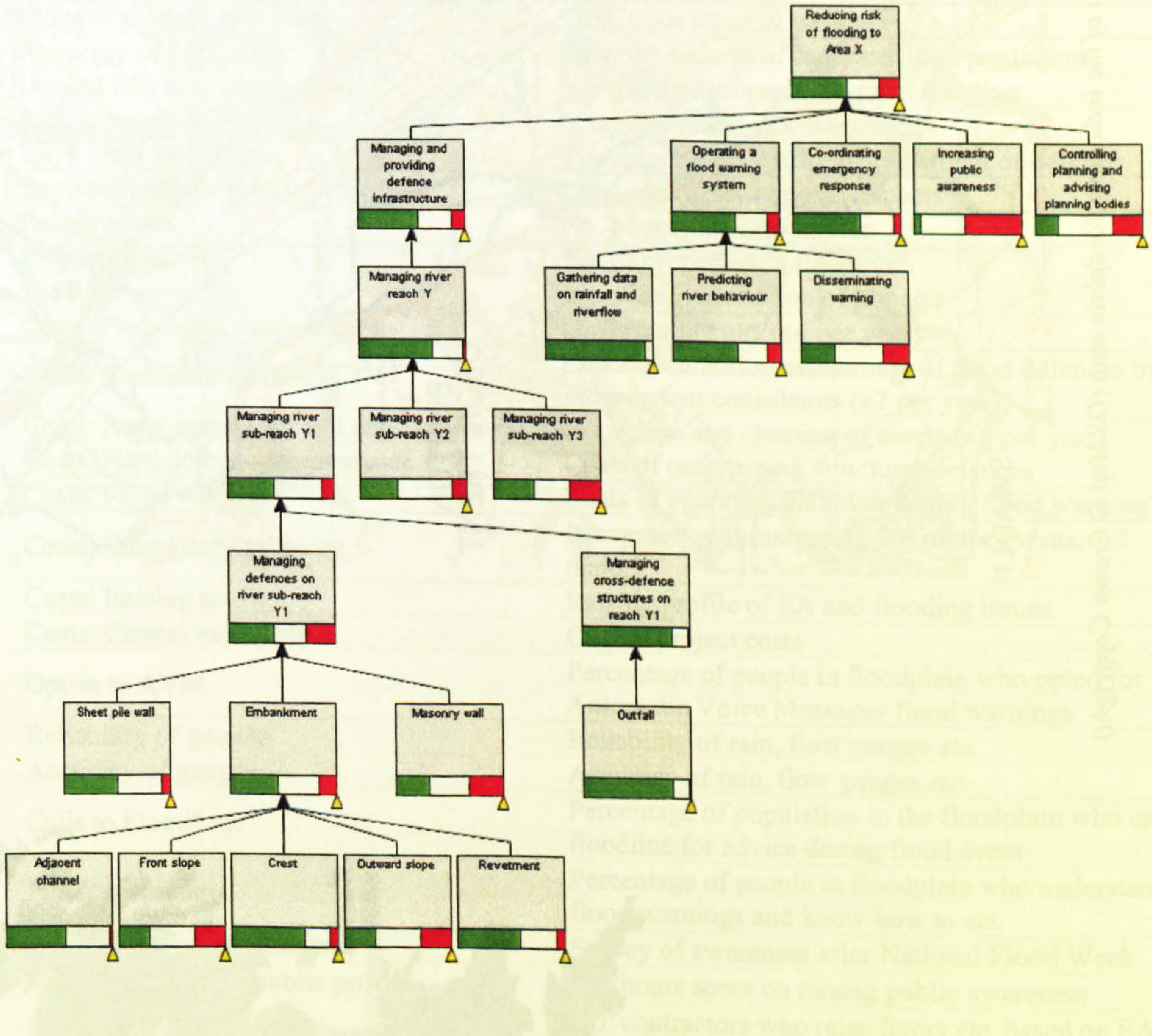


Figure 6.8 Generic process model structure for a flood defence system

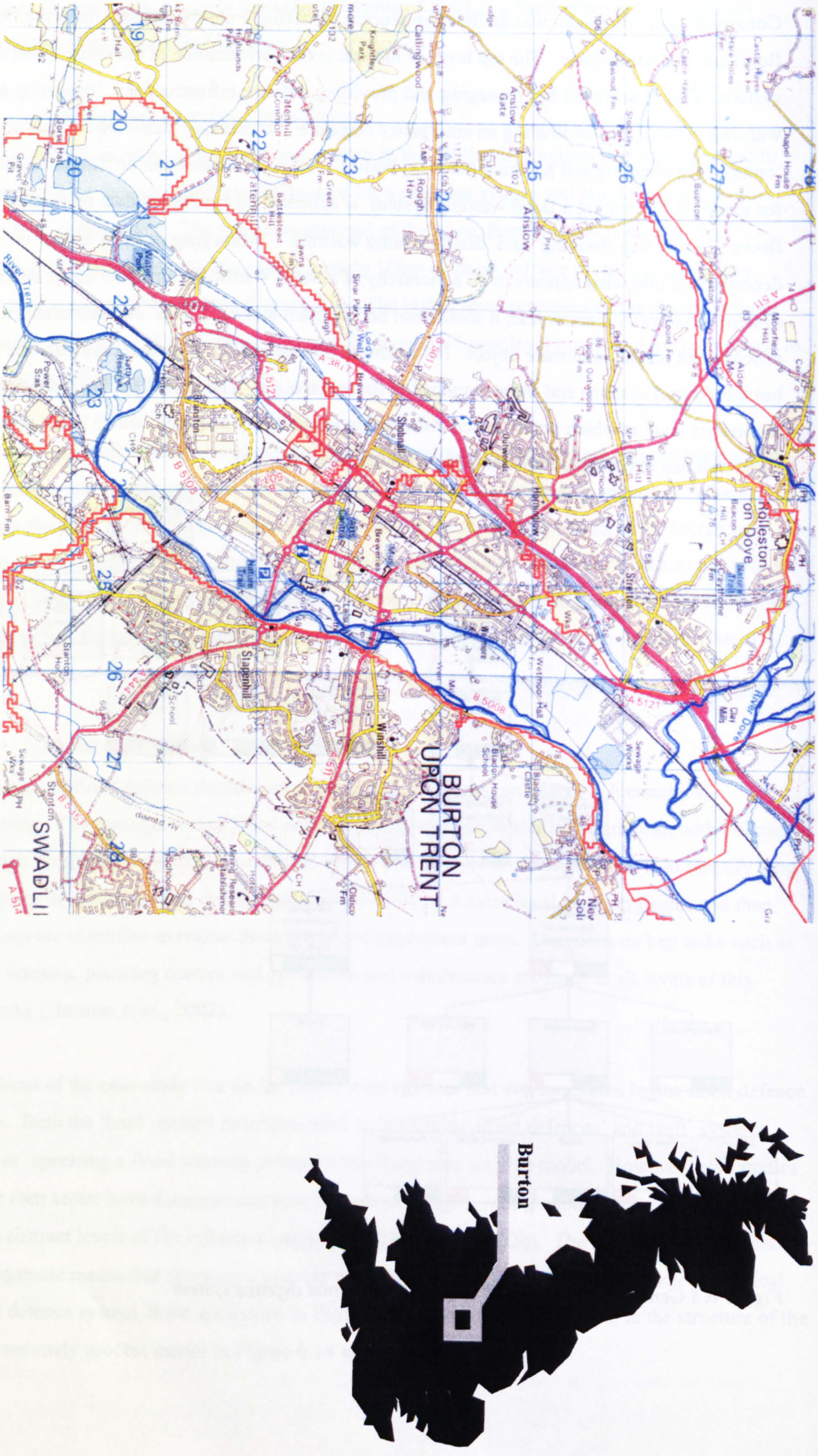


Figure 6.9 Map of Burton-upon-Trent,showing the extent of the indicative floodplain map (OS Map © Crown Copyright)

6.5.2. Populating with performance indicators and assessing their value

The size and complexity of flood defence systems means there is a large amount of evidence relating to their performance (Table 6.2).

Table 6.2 Examples of some of the many performance indicators that can be used to monitor the flood defence system

Performance Indicator	Description
Condition of structure (or element)	Visual assessment of an asset's condition.
Rotational failure towards river FoS	Factor of Safety: Rotational failure towards river
Rotational failure away from river FoS	Factor of Safety: Rotational failure away from river
Slip circle (drawdown)	Bishops Factor of Safety at drawdown conditions
Slip circle (erosion and drawdown)	Bishops Factor of Safety at drawdown conditions and some river face erosion
Overturning FOS	Factor of Safety against overturning
Sliding FOS	Factor of Safety against sliding
Thickness of sheet pile	Measure of degree of corrosion.
Flood Warning Lead Time	Time from warning to flood
Accuracy of prediction	Previous success of accurate flood prediction
Loss of life	No. of fatalities resulting from flooding
Risk assessment	Economic flood risk assessment
Days since inspection	Time since last condition assessment of defence
Environmental Impacts	Linguistic description of impacts
People at risk	No. of people in floodplain
Properties at risk	No. of properties in floodplain
Social Impacts	Linguistic description of impacts
Costs: Floodbank maintenance	Grass cutting *etc*. (x4 per year).
Costs: Condition grading	Costs of condition assessment of flood defences by independent consultants (x2 per year).
Costs: Asset inspection and debris removal	Inspection and cleaning of assets (x6 per year)
Costs: Hard defence maintenance	Costs of maintaining structural defences
Costs: Flood warning	Costs of operating and maintaining flood warning
Costs: Emergency response	In average year (assuming 3/4 minor events, 1/2 major)
Costs: Raising profile	Raising profile of EA and flooding issues
Costs: Capital expenditure	Capital project costs
Opt-in to AVM	Percentage of people in floodplain who opted for Automatic Voice Messages flood warnings
Reliability of gauges	Reliability of rain, flow gauges *etc*.
Accuracy of gauges	Accuracy of rain, flow gauges *etc*.
Calls to FloodLine	Percentage of population in the floodplain who called floodline for advice during flood event
Public awareness	Percentage of people in floodplain who understand flood warnings and know how to act.
Post flood week awareness	Survey of awareness after National Flood Week
Time spent raising public profile	No. hours spent on raising public awareness
No. of contractors who alter plans	No. contractors who raise floors *etc*. based on EA advice to reduce flood risk
Local Authorities who refuse planning	LA planning applications that have been refused/ required modification after EA advice

This evidence needs to be considered in a holistic and structured manner that allows a decision-maker to make informed choices based on the performance of all aspects of the system.

Performance indicators are entered into the model at the level within the system they are measured. Examples of evidence and appropriate value functions are given below.

Table 6.3 Examples of performance indicator values and value functions

Performance indicator	Value function	Measured Value	Performance
Crest condition	Linguistic	Good (Medium)	[0.77, 1.00]
100 year flood	S-Shaped	45.03m (±0.02m)	[0.75, 1.00]
200 year flood	S-Shaped	45.03m (±0.05m)	[0.10, 1.00]
1000 year flood	S-Shaped	45.03m (±0.10m)	[0.00, 0.99]
Rotational failure FoS	S-Shaped	1.03	[0.15, 0.91]
Maintenance costs	Linear	£4000 (±2000)	[0.99, 1.00]
Capital costs	Linear	£1.2m (±£0.1m)	[0.89, 1.00]
Flood warning lead time	S-Shaped	120min (±15min)	[0.88, 0.96]
Loss of life	Stepped	0 people	[1.00, 1.00]
Accuracy of gauges	Linear	90% (±10%)	[0.75, 1.00]

Each structural element has associated with it a qualitative measure ranking a defence's visual condition between "very poor" and "very good". Each element may also have other specific indicators; for example, crest has a measure of height. Appropriate value functions are applied to the performance indicators. The condition grade is treated as a linguistic descriptor with the five point scale which maps conveniently on to the linguistic value function five point scale as demonstrated in Figure 6.5.

Crest height is more complex as its performance can only meaningfully be assessed when compared with hydraulic loading. Flood defence performance is best described by considering a whole range of loading conditions (Chapter 5). To capture this, the crest height was valued against the 100, 200 and 1000 year flood levels. This is shown in Figure 6.10. The performance of the crest is seen to decrease and become more uncertain against more extreme floods. A higher weighting is applied to the performance against the 100 year flood to represent both the fact that it is more likely to occur, but also that this is the flood the defence was built to withstand. Where a complete distribution of loadings is available, it may be preferable to calculate the probability of overtopping directly. However, valuing the crest height against individual flood levels allows both the performance of the defence over a range of loadings and the increase in uncertainty associated with estimating the more extreme flood levels to be captured in a satisfactory manner.

There are many performance indicators relating to cost. These include costs of defence maintenance, cost of flood warning and other costs such as cost of raising awareness to flooding. At a high level, these costs will be an aggregation of costs at lower levels within the organisation. Costs can be evaluated in terms of performance by comparison with budget. For example, if defence maintenance costs for a river reach are £10,000 (±£2000) per year out of a total maintenance budget of £2,000,000 then that reach will have a relatively high level of economic performance. The size of this maintenance budget can be measured against the total budget to allow the effects of increasing maintenance at a loss of investment elsewhere in the system.

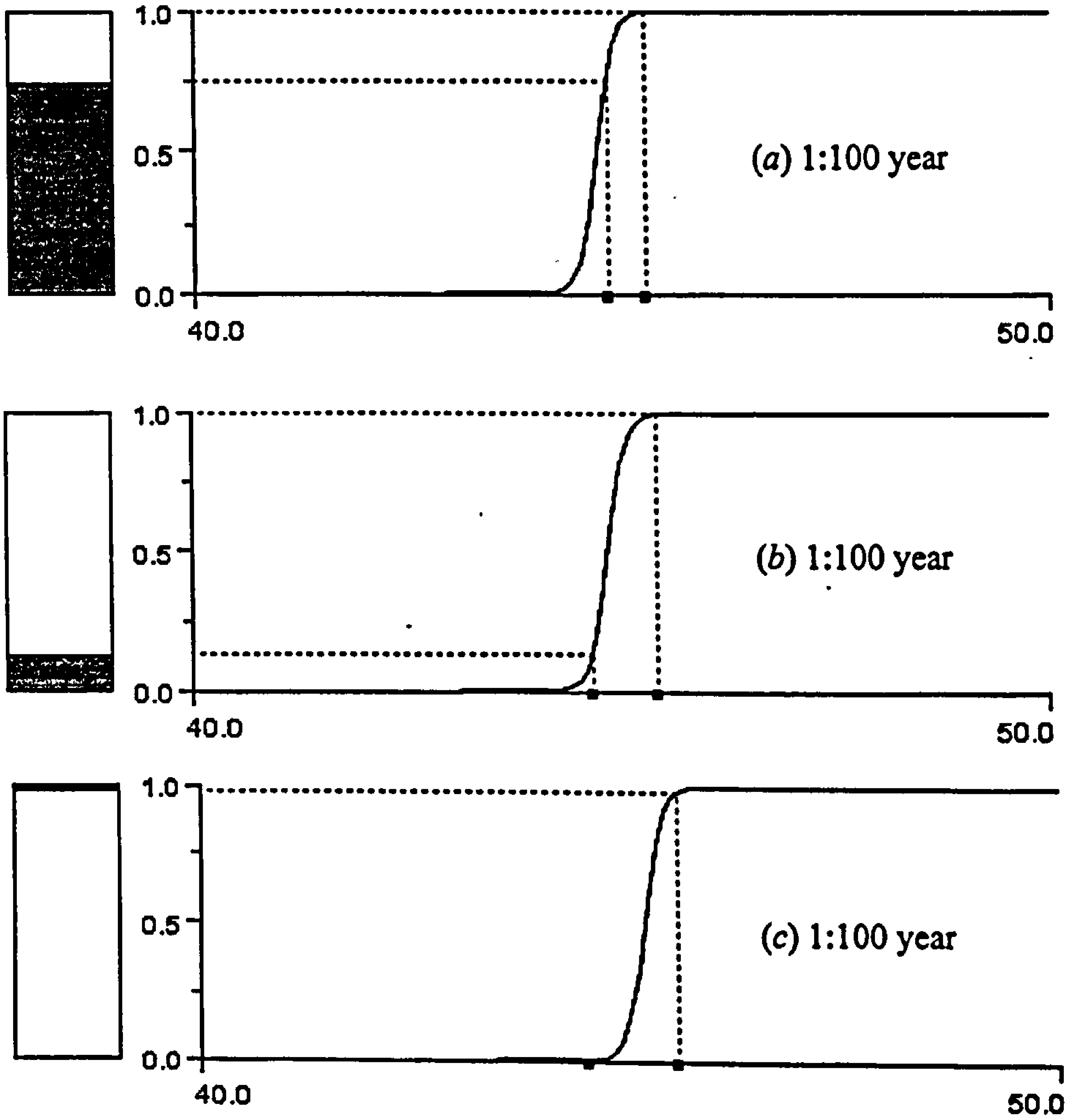


Figure 6.10 Mapping the performance of a defence crest through a value function for the (a)1:100 year, (b)1:200 year and (c)1:1000 year flood events

Softer indicators such as public awareness to flooding are of great relevance to the overall objective of reducing flood risk. Several performance indicators were adapted from the results of surveys of members of the public in the flood area. One such indicator, percentage response to flood warning (*i.e.* percentage of public who took action to reduce damages after receiving a flood warning), was mapped through a concave value function (Figure 6.3(c)). This reflects an increase in performance associated with a greater public response, but also that the performance gain increases as more people respond to flood warnings.

6.5.3. Risk and fragility as performance indicators

Chapters 4 and 5 introduced a risk assessment methodology and a condition characterisation methodology. Both of these act as important performance indicators for a flood defence system.

A risk assessment provides an overall estimate of economic risk for the flood area, being Burton-upon-Trent for the case study model. A risk assessment also provides indicators of social risk. This can be expressed in terms of the number of people at risk or a measure of social vulnerability (Tapsell *et al.*, 2002) that identifies people who will find it harder to cope with a flood event. These both act as a measure of performance at the highest process in the model as it provides a measure of the performance for the whole of the Burton-upon-Trent flood defence system. Were the model to be extended to include regional or national processes, aggregated measures of flood risk for river catchments and on a national basis could be included. Economic flood risk is mapped through a linear value function and the risk for a flood system such as Burton is valued against the overall potential consequences of flooding. Consequently a decrease in flood risk to the town will increase the performance of the flood risk indicator. However, it should be noted that a reduction in risk is most likely associated with a corresponding decrease in performance resulting from the increased cost. The benefit of a performance-based methodology over a purely risk-based decision is that additional benefits (or losses) to the population or the environment such as amenity can also be considered.

A risk assessment also produces a performance indicator for specific defences in terms of their contribution to the overall risk. This is mapped through a linear value function and is valued in terms of its contribution towards the risk for the whole system. The appropriate level of including risk assessments in a hierarchical process model of the flood defence performance is shown in Figure 6.11.

Chapter 5 introduced a new condition characterisation approach that used fragility curves to act as a measure of structural performance. These can be incorporated into the model using one of two methods. The first employs a similar method as that used to evaluate defence crest performance can be employed. The upper and lower bound of conditional failure probability are taken from the fragility curve for the 100, 200 and 1000 year flood and mapped through a linear value function. The lower and upper bounds on failure probability $[Pf_l, Pf_u]$ therefore correspond exactly to $[S_n(E), S_p(E)]$. These items of evidence are then weighted against each other in the ratio 1:0.5:0.1 to represent the relative frequency of the events and therefore their relative importance to defence performance. A figure of merit based purely on defence condition is therefore:

$$FM=\{[Pf_l(100), Pf_u(100)]\times1.0+[Pf_l(200), Pf_u(200)]\times0.5+[Pf_l(1000), Pf_u(1000)]\times0.1\}/3 \quad (6.22)$$

The second method requires a loading distribution, l, which is combined with the defence fragility function, F_R, using Equation 6.23 to produce lower and upper bounds on defence failure $[Pf_l, Pf_u]$.

$$Pf=\int_0^\infty F_R(l)dl \quad (6.23)$$

Again, these bounds are mapped through a linear value function onto a figure of merit to produce a representation of the total structural performance of the defence.

Structural degradation and climate change scenarios are modelled by altering the performance indicators to represent the change in fragility or loading.

Both risk and fragility provide useful information on the performance of the flood defence system. However, care should be taken when entering them into the model as some of the evidence from which they are constructed is already included in the model. For example, the high level risk assessment methodology in Chapter 4 relies on the defence condition assessment. This means there is a high degree of dependency between the two pieces of evidence being used to assess system performance and it is important to adjust the dependency values appropriately to ensure that the performance is not overestimated.

6.5.4. Assigning necessity, sufficiency and dependency values

It is very rare that exact dependencies have been established for sub-systems within the flood defence system. Neighbouring defences often exhibit a strong dependency, but this dependency will tend to zero over a long distance. CUR and TAW (1990) identify methods of calculating the correlation of strength between sections of defences, however these methods have not been applied in England and Wales due to the difficulty and cost of obtaining information on dependencies which involves dense and often destructive testing of flood defences. Dependency between the elements of defences is dominated by the likely failure mode of the structure. Assessment of defence strength normally results in bounds of failure being calculated based on the assumption of total dependence or independence rather than the establishment of the dependency between failure modes.

In the light of no better information, a default value was set for the dependency, necessity and sufficiency between flood defence elements and neighbouring defences. A dependency of 0.5 was chosen to reflect the fact that there is clearly dependency between neighbouring defences and individual defence elements. The necessity is also 0.5, this represents the fact that whilst failure or a defence element results in a considerable loss of performance of the defence it will not necessarily fail completely. The sufficiency of defence elements is 0.2. This represents the fact that the structural elements do not completely represent the defence's sub-systems as much of the defence is unseen, there is also no evidence to suggest that any particular element is consistently more important than another in determining the defence's performance. This also rings true for individual defences within a defence system which have been assigned the same values of dependency, necessity and sufficiency.

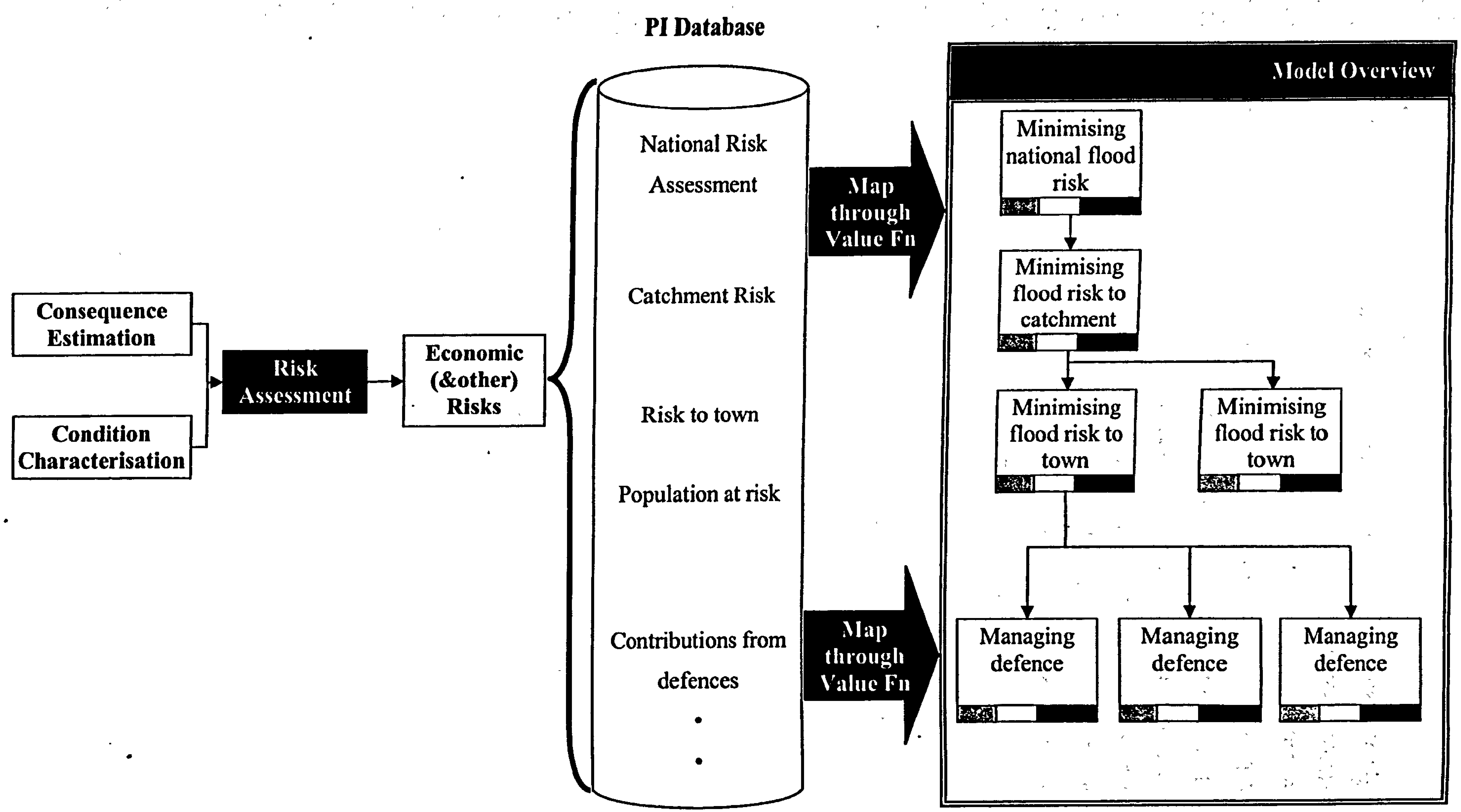


Figure 6.11 Integrating a tiered risk assessment with a hierarchical process model of a flood defence system: An estimate of consequence and assessment of defence condition are used to estimate risk. Risk values are entered into the performance indicator database and mapped through value functions as described in Section 6.5.3 where they are weighted against other evidence in the process model

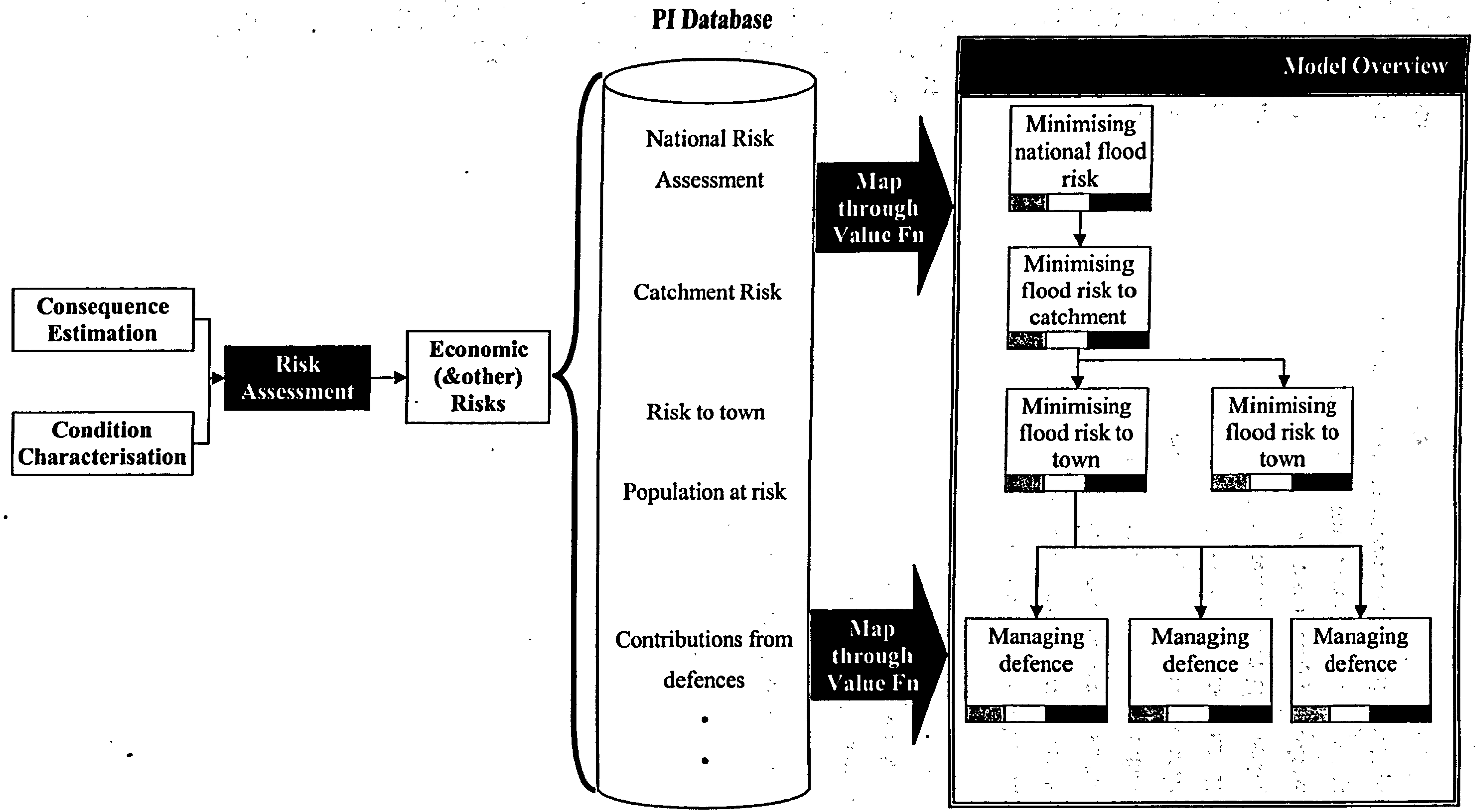
The relationship between higher level processes is even more subjective. After the model was constructed and populated with performance indicators, default values were assigned to dependency, necessity and sufficiency and altered to reflect what experts thought they should be and how they perceived the high level processes to be influenced by changes in performance at lower levels within the hierarchy.

Whilst assigning dependency values can sometimes be formalised with mathematical analysis, these three measures rely heavily on expert judgement. This should be recorded in an appropriate manner so that it is open to scrutiny and review. The sensitivity of these three measures can be explored by the modeller, however, a suitable semantic interpretation of these numerical measures needs to be established so values can be more readily assigned.

6.5.5. Scenario testing

Once a model has been constructed, it can be used to test management intervention scenarios. Five categories of scenario of interest to a flood defence manager were identified; maintenance, capital works, climate change, policy change and evidence collection scenarios.

To evaluate how a maintenance scenario may change system performance, defences in the system were subjected to increased maintenance activity, resulting in all defences in "medium", "poor" and "very poor" condition being increased to "good" condition. Improved condition would be expected to raise system performance, whilst the corresponding increase in cost would decrease performance. The effect of maintaining the defences also gives more confidence in the condition assessment, therefore the confidence is increased from "medium" to "high". Altering the relevant performance indicators allows the effects of the scenario to be explored. The original performance of the flood defence system is shown in Figure 6.12 as being [0.33, 0.60] whilst Figure 6.13 shows the performance of the flood defence system after increased maintenance of the defences as being [0.37, 0.60]. The consequence is therefore an increase in the overall performance of the flood defence system and a decrease in the associated uncertainty. This increase in performance demonstrates that the increased maintenance expenditure will improve the performance of the flood defence system. This can be used by a decision-maker to justify cost of maintenance. The increase in performance can also be compared to other scenarios, for example, whether it would be better to spend less and increase the condition of defences only to "medium" rather than "good". It might be expected that a significant increase in maintenance would have a much greater impact on the performance of the system. This will not be the case as the whole system is being modelled and its performance is dependent on the performance of other subsystems such as flood warning and emergency response. The model that has been constructed considers only part of a larger system (the entire river catchment) and therefore this increase in performance would have to be considered in the context of the rest of the system, for example, whether the resources invested on maintenance could be better used up or downstream.



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Capital works scenarios can be tested by altering the model itself. Capital works involve the construction, replacement or significant upgrading of existing defences. This is modelled by adding or replacing processes in the model.

Climate change scenarios can be modelled by adjusting the loads on the system. Predominantly this will alter the performance of the defence crest resulting in a (likely) decrease in performance, coupled with an increase in uncertainty. As described in Section 6.5.2, the loads, represented by the river depth are used to value the performance of the crest height, an increase in river depth for the 100 year flood will result in a loss in performance.

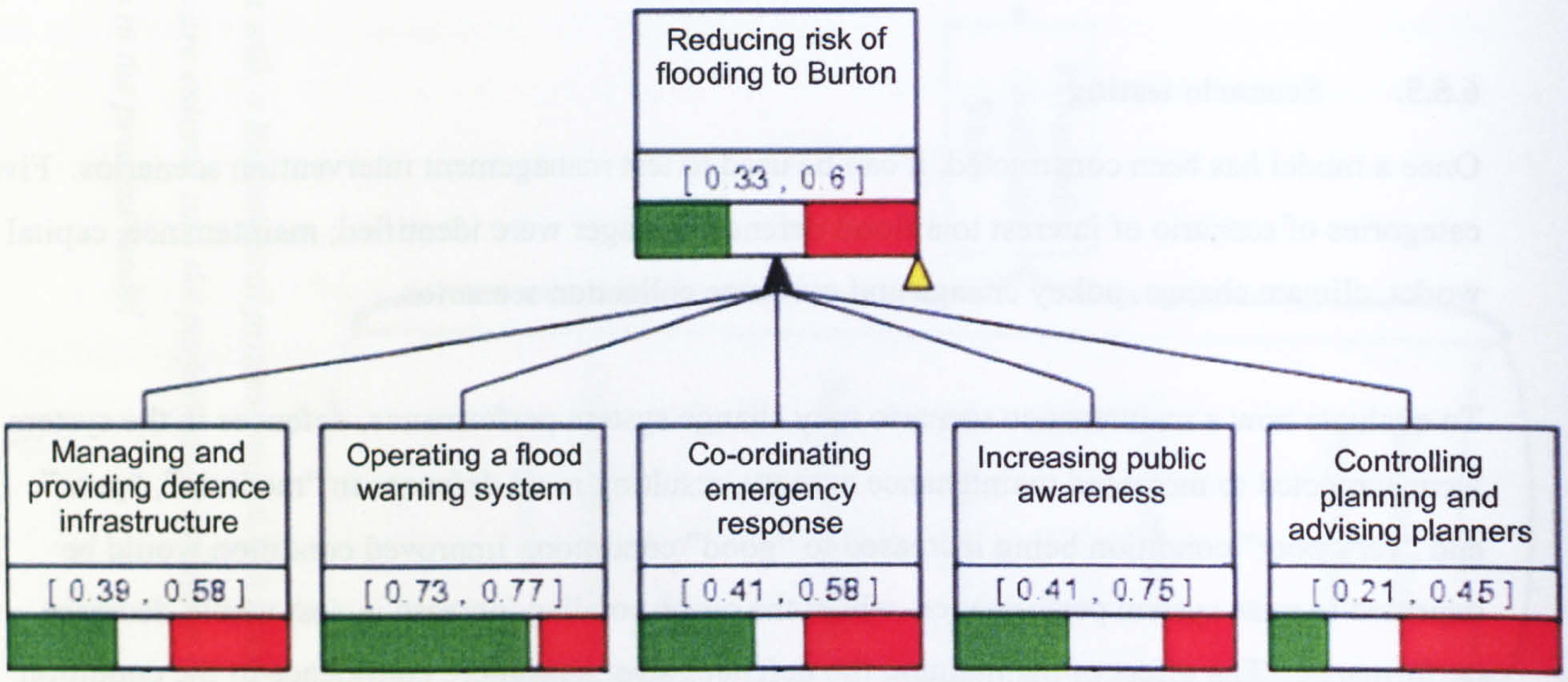

Figure 6.12 An overview of the high level processes of the Burton flood defence system

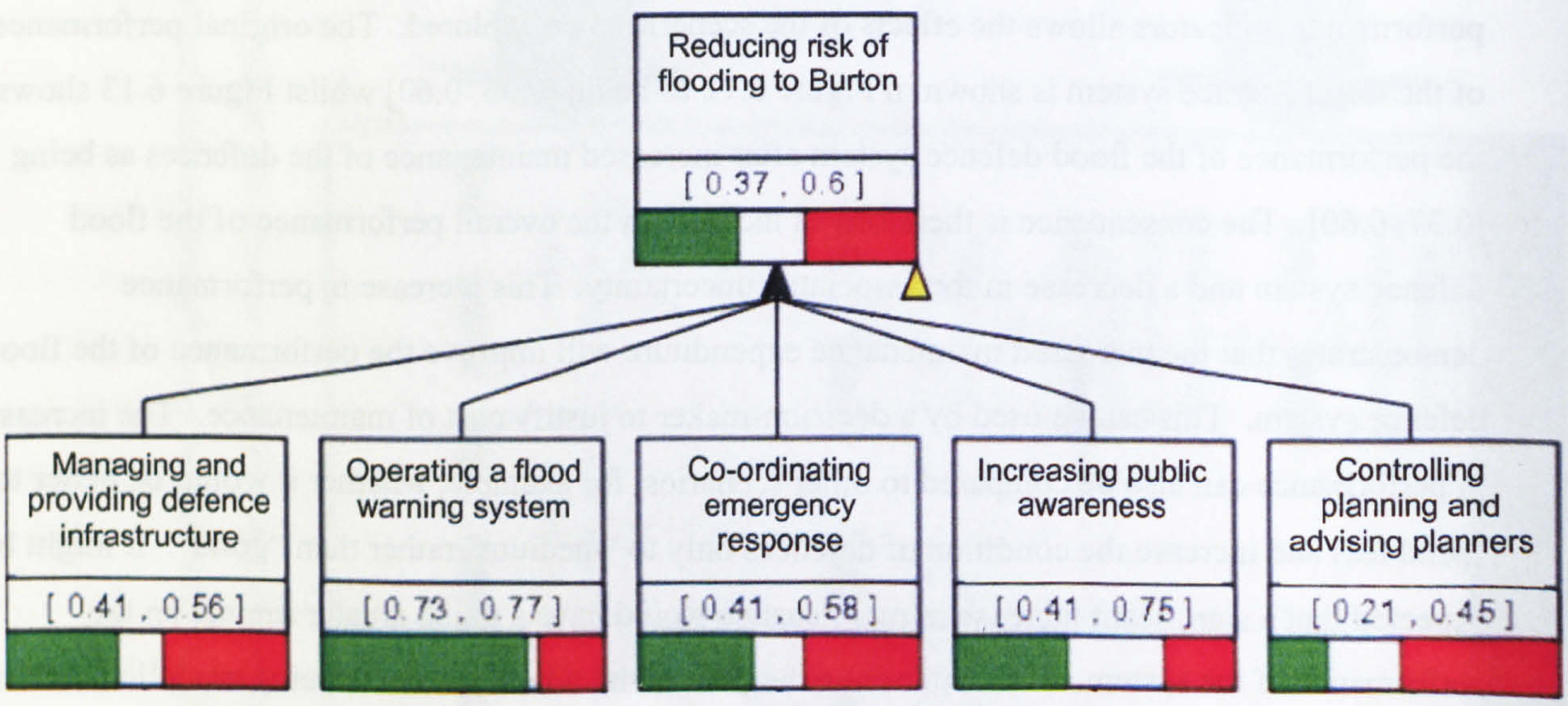

Figure 6.13 An overview of the high level processes of the Burton flood defence system after maintenance improved the condition of the defences

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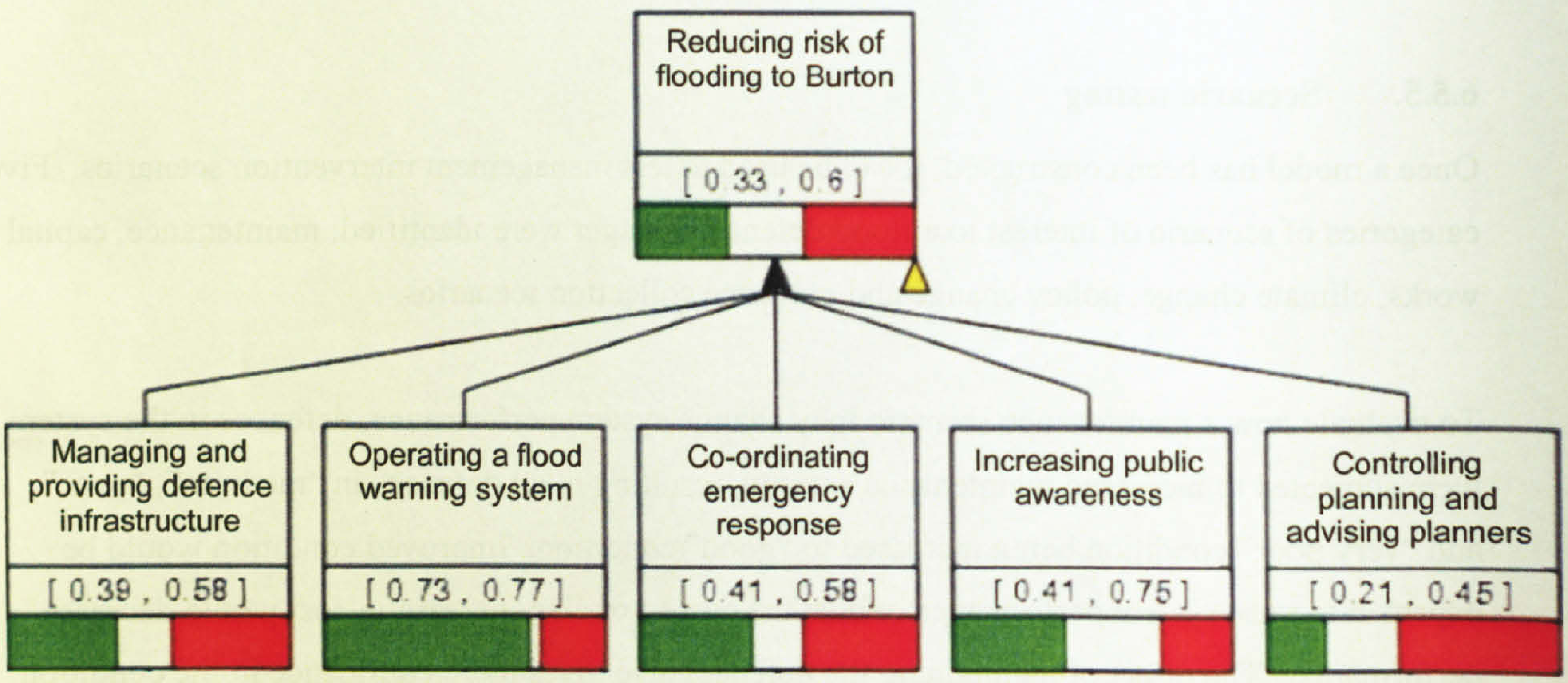

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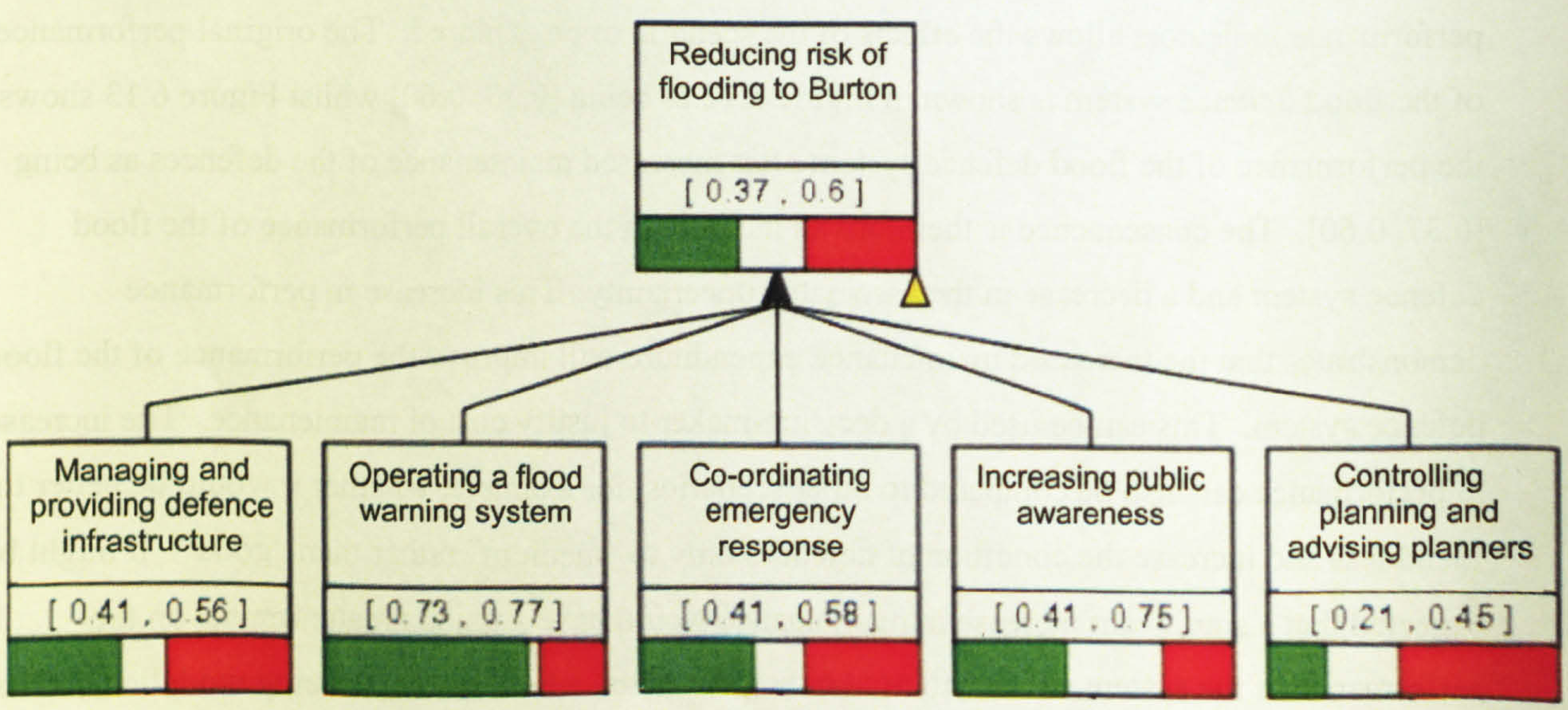


Figure 6.13 An overview of the high level processes of the Burton flood defence system after maintenance improved the condition of the defences

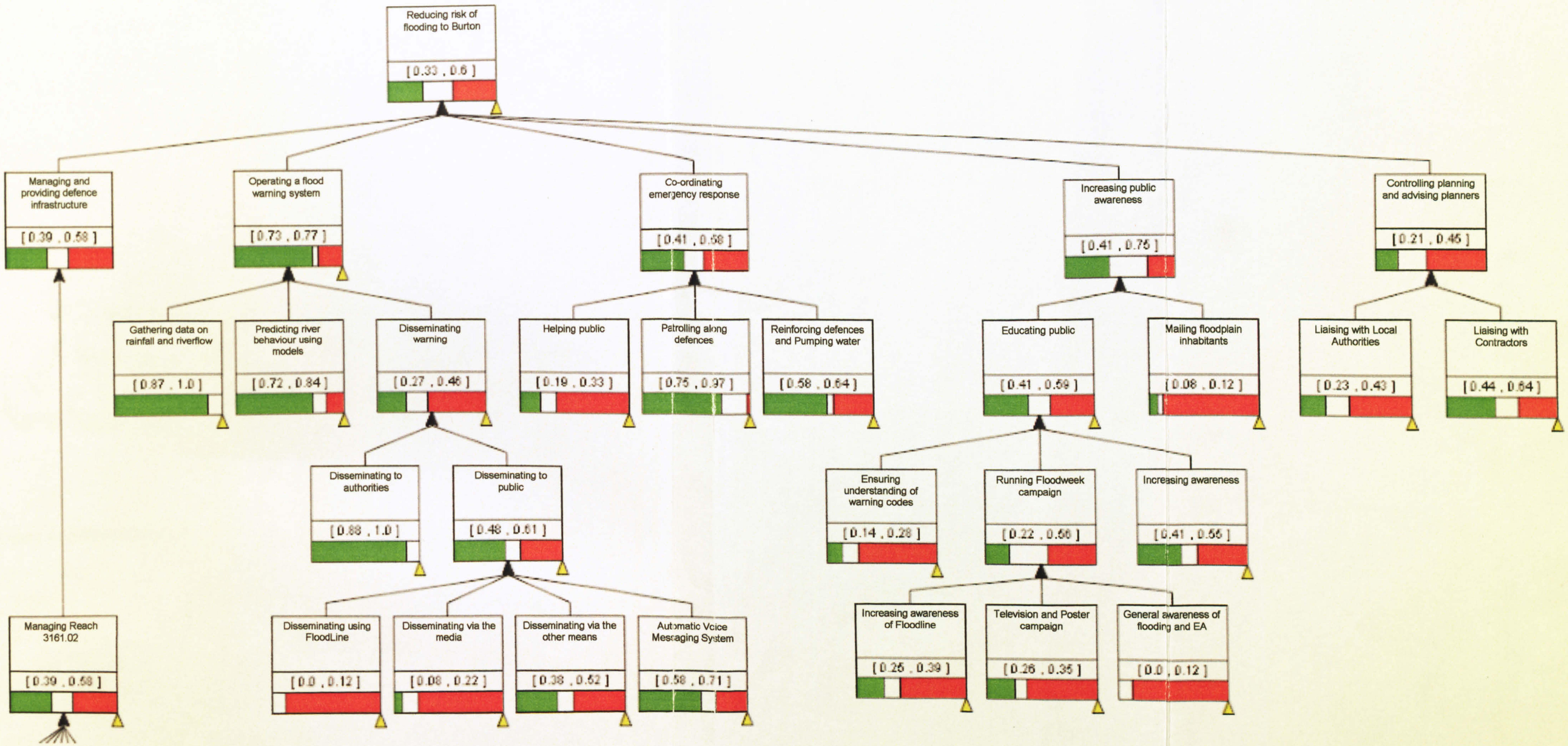
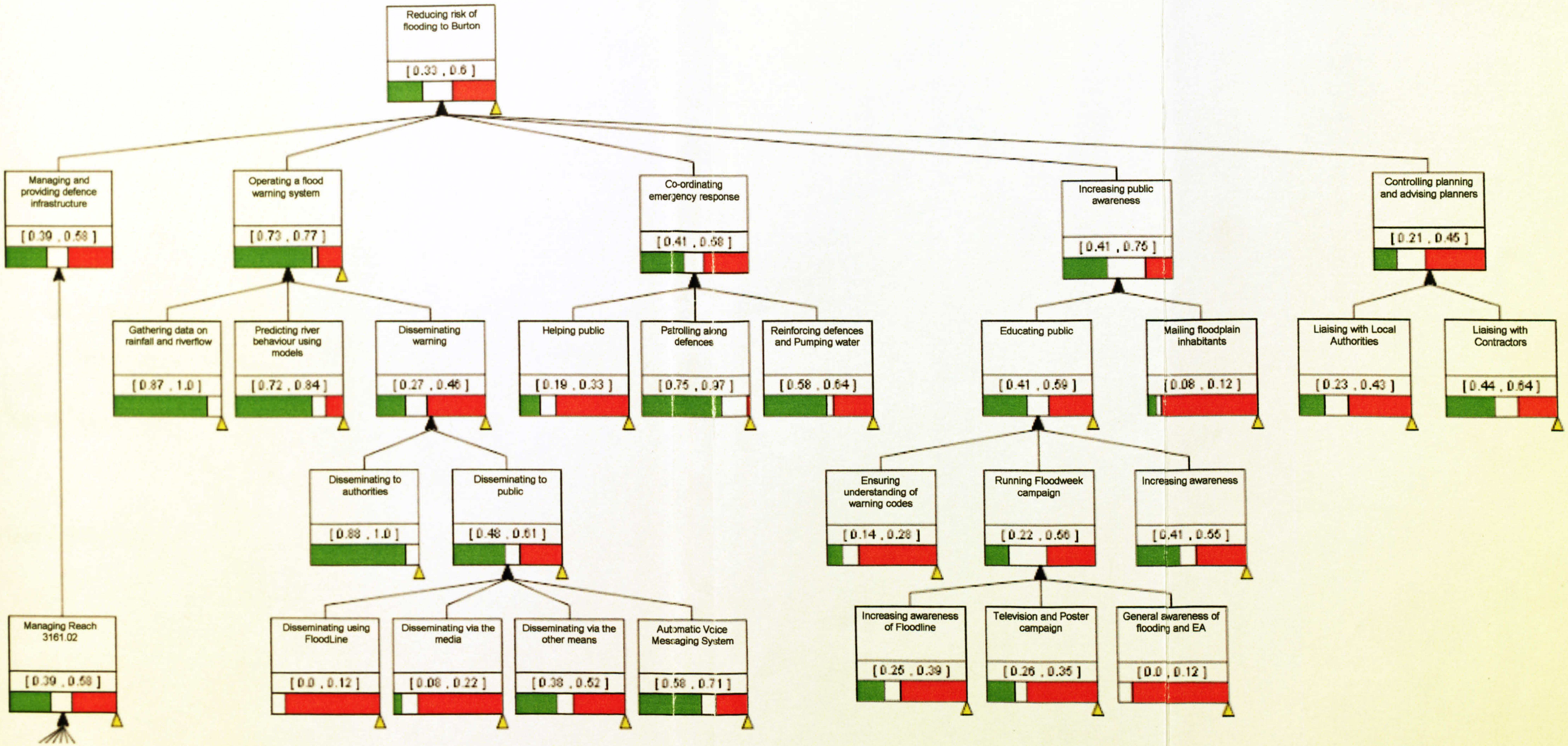


Figure 6.14 Hierarchical process model of the Burton-upon-Trent flood defence system (Upper section)

In Figure 6.15(a) and (b) the structural elements of the flood defence are annotated as follows; FI represents the Inward Face of the defence; FO the Outward Face; FC the Crest; BE the berm; and CS the Channel Side.



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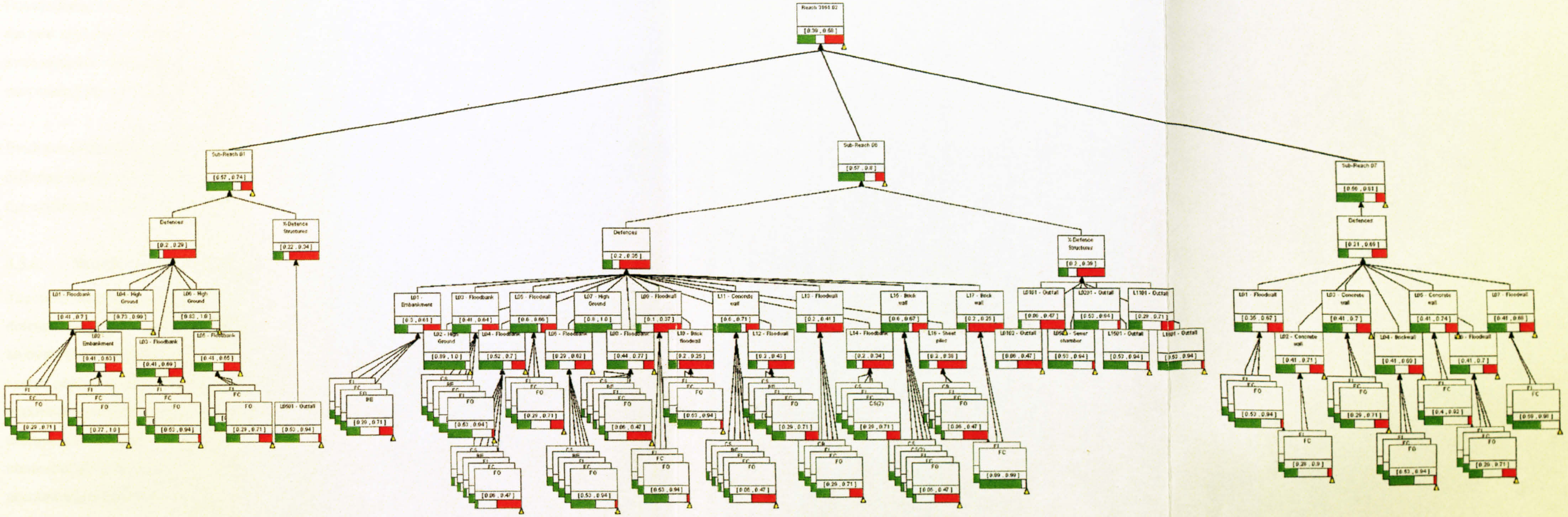

Figure 6.15 Hierarchical process model of the Burton-upon-Trent flood defence system (Lower section: Part I)

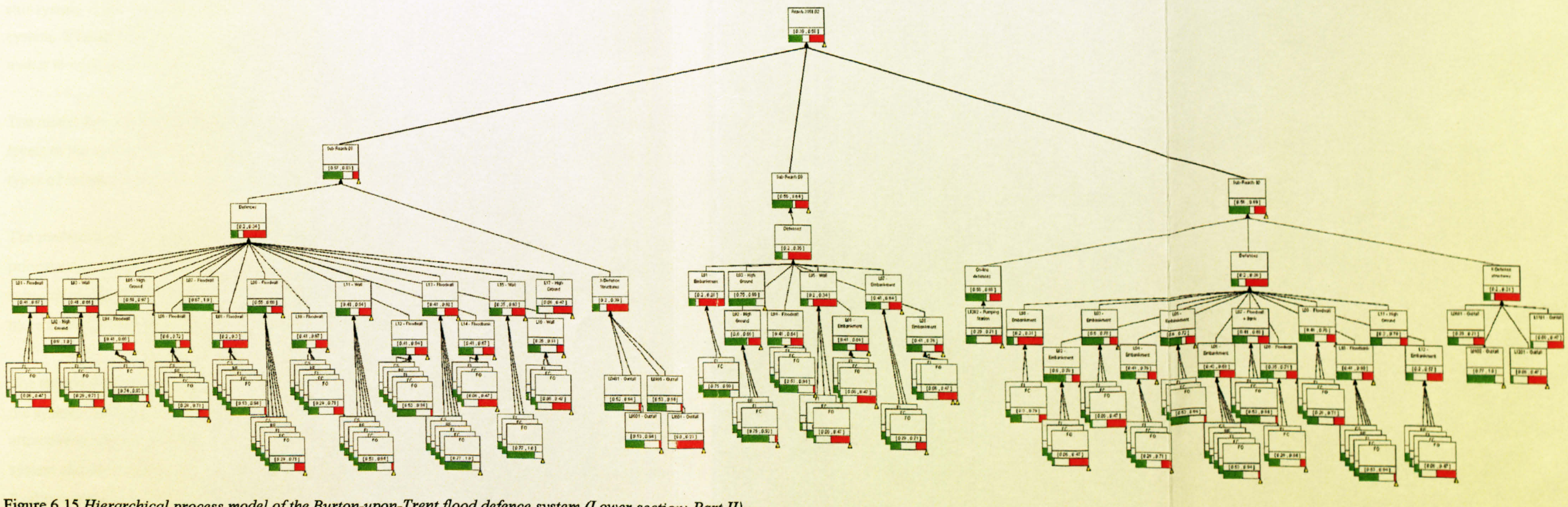

Figure 6.15 *Hierarchical process model of the Burton-upon-Trent flood defence system (Lower section: Part II)*

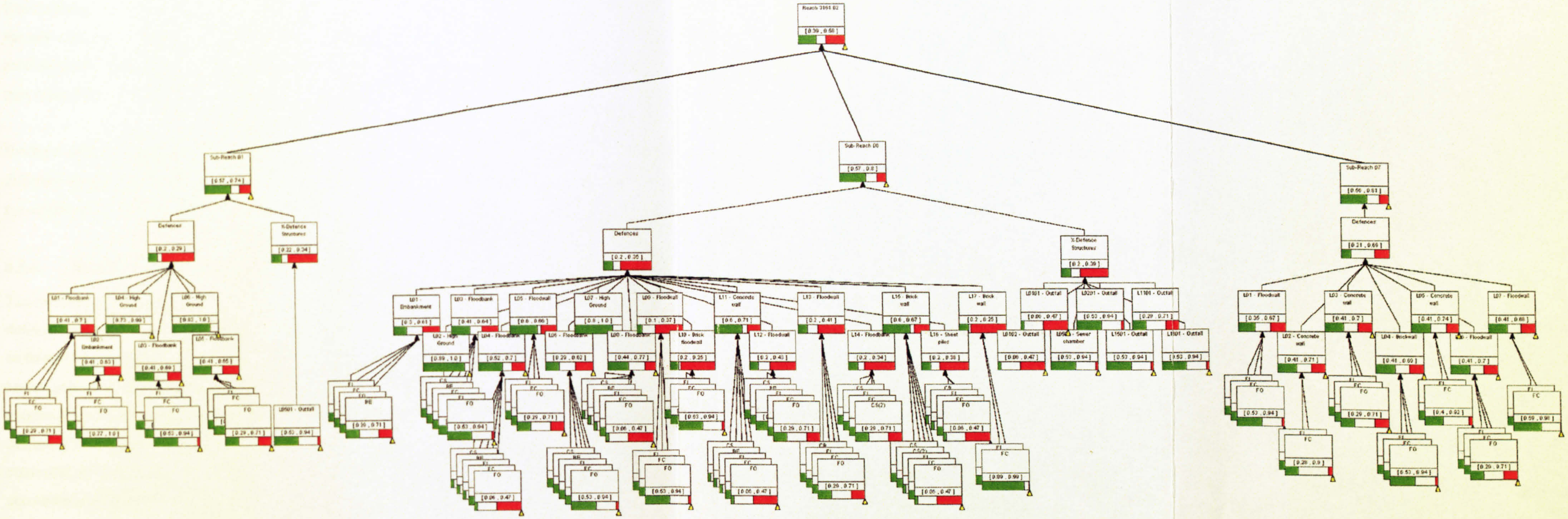

Figure 6.15 Hierarchical process model of the Burton-upon-Trent flood defence system (Lower section: Part I)

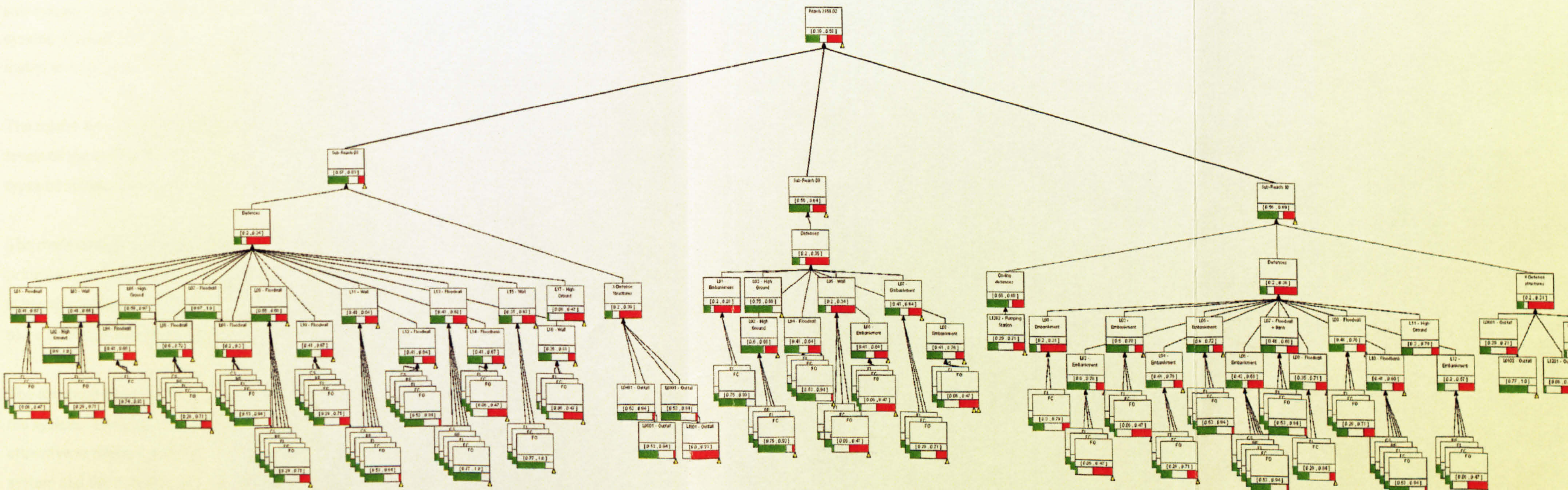

Figure 6.15 *Hierarchical process model of the Burton-upon-Trent flood defence system (Lower section: Part II)*

Policy change scenarios require the alteration of the model or its performance indicators to reflect the new organisational priorities. For example, a simple change in fund allocation, perhaps increasing the maintenance budget and reducing the capital works budget, requires the appropriate cost related performance indicators to be altered.

Evidence collection scenarios can be used to test the advantages of collecting (or ceasing to collect) different sources of evidence. This allows the decision-maker to weigh the reduction in uncertainty the evidence provides against its cost.

6.5.6. Benefits of the methodology and model

The methodology and model of the flood defence system provide many clear benefits to a flood defence manager. The case study has resulted in a model of a flood defence system that provides an overview of performance at a range of levels within the system by ordering a large number of assets, processes and evidence in a manner never previously achieved. In particular, the model maximises use of the results of a risk analysis by incorporating them as performance indicators at multiple levels in the system description. However, this analysis is incorporated into the broader context of all of the sources of evidence and values that the asset manager will wish to take into account providing a useful and relevant flood defence management tool.

The figure of merit provides a clear representation of the amount of uncertainty associated with the sub-system. This uncertainty is a reflection of the lack of knowledge of the behaviour of the system. Further investigation can establish the sources of this uncertainty, allowing the decision-maker to target resources more efficiently.

The model acts as a platform for exploring potential investment decisions and their impacts on all levels of the system. This has been demonstrated by testing a maintenance scenario, but other types of scenario are suggested in Section 6.5.5.

The methodology provides a transparent and auditable mechanism for exploring, justifying and prioritising different types of investment decisions at all levels within the system. Clearly some subjective judgements are required, but these are open to scrutiny and discussion. This provides a more auditable and transparent mechanism than that presently in place.

The approach proposed to capture performance uses process modelling to capture the behaviour of the system. The graphical representation of performance in terms of an interval has proven attractive to decision-makers. It is straightforward to explain, yet enables the complexity of the system and the richness of the evidence to be captured. A key benefit of process modelling was to facilitate communication between decision stakeholders focused on different areas of the flood defence system. Many decision-makers had previously considered only the effects of their decisions on a limited part of the system and were in some cases initially surprised that decisions they make can influence the performance in different parts of the system. Using the model, engineers are able to communicate the importance of maintenance to the overall performance of the system, however, they can also be shown how a public liaison officer can also increase system performance by educating the public thereby providing a platform for comparing non-engineering based flood risk management techniques with more traditional approaches. Managers are therefore provided with a holistic view of the system and are able to view the relative importance of sub-systems to its overall performance. The model can also be used to help externalise information and decisions to organisations and the general public.

Sceptical decision-makers, whilst recognising the benefits of the approach, have pointed out that model construction and maintenance represents yet another time-consuming activity. The implementation of the methodology in the form of the user-friendly PERIMETA software has helped to reduce this. However, the balance between the cost (measured in time) and benefit of model construction will shift in favour of this approach as it becomes more integrated with data initiatives such as the National Flood and Coastal Defence Database (Linford *et al.*, 2002). Despite this, the very act of modelling the system provides immediate benefits in its own right as decision-makers are forced to think in a way they had not previously about the processes they and others enact and the information that is available and its importance.

6.6. SUMMARY

A new approach to assessing the performance of flood defence systems has been proposed to support decision-making. The system processes are modelled hierarchically. Evidence of performance of the flood defence system is mapped through a value function to produce a non-dimensional measure of performance. Evidence is propagated through the system using interval probability theory. The performance of each sub-system is represented by a figure of merit that provides a useful overview of the evidence of successful and unsuccessful performance of each sub-system.

The methodology has been successfully tested on a case study for Burton-upon-Trent. A possible maintenance scenario was suggested to demonstrate how the methodology could be used to support investment decisions.

The risk assessment and condition characterisation methodologies proposed in Chapters 4 and 5 provide useful evidence of performance of the flood defence system. It has been shown that these can be integrated into the model hierarchy thus providing a useful means of making decisions based on flood risk within the wider context of system performance.

The needs for an improved decision-support methodology that were outlined in Chapter 2 have been satisfied. A transparent and auditable decision-making methodology has been supplied that can be used to support decisions by considering the performance of system processes and the uncertainty associated with evidence of performance.

7.1.1. Flood risk assessment

A review of flood risk management in England and Wales identified flood risk assessment as providing a powerful basis for decision-making. A quantitative flood risk assessment can be used to support the appraisal of scheme design, policy options, resource allocation and as a measure of performance of the substantial annual investment in flood management.

To reflect the many different types of decisions made at various organisational levels within the flood defence management structure, a tiered probabilistic risk assessment methodology that builds on previous research by Meadowcroft *et al.* (1996) was proposed. Each level of risk assessment provides an increasingly more accurate estimate of flood risk. This allows the flood defence manager to commit resources applied to the risk assessment appropriate to the scale and importance of the decision. A measure of national assessment of flood risk need only be relatively coarse, whereas the designer of a flood defence scheme will require an accurate estimation in order to optimise the scheme design. Three levels of risk assessment have been proposed, a *High Level* analysis that can be performed on a national scale, making use only of data available nationwide. The *Intermediate Level* incorporates additional information on loading, floodplain topography and defence structure to provide a risk assessment on the scale of a sub-catchment or coastal sub-cell. The *Detailed Level* uses information about the composition of the defences and a much more detailed study of their proneness to failure to provide the most accurate flood risk assessment at a scheme level. Whilst increasingly more accurate results are provided, at all levels of assessment:

- uncertainties in the QRA are represented as intervals,
- defence condition is described over a full range of loads using fragility curves, and,
- a GIS based impacts assessment is employed.

The High Level risk assessment methodology has been described in detail. An example implementation for the river Parrett and Bridgwater Bay has demonstrated the applicability and potential of the methodology as a tool for decision-support. Climate change and maintenance scenarios demonstrate how the methodology can be used to support decisions on national flood defence investment policy. This method has now been applied for the whole of England and Wales (HR Wallingford *et al.*, 2003) the results of which have been summarised and discussed in Appendix G. The high level method provides a number of benefits over the previous national scale flood risk assessment by Halcrow *et al.* (2001) because:

- defence failure probability is calculated as a function of load using fragility curves rather than as a point value, thereby providing a more complete overview of defence performance,
- the upstream and downstream extent of flooding is estimated instead of assuming that a defence inundates to some degree all the floodplain on the same reach, thereby allowing defences to be associated with the parts of the floodplain they protect,

- multiple defence failures are considered, allowing the compound effects to be analysed, thereby considering all possible failure scenarios that can result in flooding of a given impact zone, and,
- the depth of flooding in each impact zone for each flood event is estimated instead of using event weighted damages for each property, thereby providing more accurate damage estimation.

Despite these improvements, there are still a number of limitations with this national scale risk assessment. Some of these are addressed at the intermediate level risk assessment. The intermediate level is more process-based. Loadings are expressed probabilistically, defence performance is measured in terms of more parameters and inundation models are used to establish more accurate damage estimates. This has not, as yet, been implemented but the methodology has been fully described. The most detailed level will use continuous simulation but is, at the time of writing, less well developed and has therefore only been outlined.

7.1.2. Condition characterisation

Present approaches to condition characterisation in England and Wales rely on expert judgement to describe the defence as being in a condition that ranges from "very poor" to "very good". Probabilistic methods that are described in national guidance for economic appraisal (DEFRA, 2000a and 2000b) rely on expert judgement to estimate the total failure probability of the defence. A method using fragility curves to separate the loading assessment from the strength assessment has been proposed. Integration of the fragility curve over a probabilistic description of loading enables the failure probability of the defence to be calculated.

The conditional probability of failure over a complete range of loadings (such as wave height or water level) can be estimated using first order reliability methods. Probabilistic methods such as these are not new to flood defence engineers, however their uptake in England and Wales has been limited partly due to a lack of necessary data. The other major reservation upheld by many engineers is that the uncertainty associated with estimating many parameters is not amenable to probabilistic treatment. Much of the evidence associated with condition characterisation is often vague and this type of uncertainty is better expressed as an interval measure or a fuzzy set. The reliability methods have therefore been adapted to capture this type of uncertainty and propagate it through the assessment of failure probability so the flood defence manager can consider it in the decision-making process.

Examples using rock armour revetments and sheet piles have been provided to support the methodology. These demonstrate how uncertain information can be incorporated into the condition characterisation. System failure bounds are used to generate upper and lower bounds of failure of a structure when considering multiple failure modes. Structural degradation, which is of great importance to a strategic planner, can also be modelled probabilistically. This is achieved by unsuitable for application within an organisation such as the Environment Agency. A DSS should include:

- A DSS navigation front-end (representing the structure of the system),
- A GIS interface to show the spatial distribution of flood and erosion risks (this thesis contributed towards this by developing a quantitative flood risk assessment methodology),
- A dynamic link with key models and databases (this will include models and datasets that are likely to be used by the majority of users, or needed for decisions that need to be made in real-time such as flood forecasting),
- Decision and uncertainty analysis tools (Chapter 6 presented a performance-based decision-support methodology),
- An inventory of other available data and models (models and datasets that do not need to be accessed immediately or by all users),
- Guidance on model selection and use, and,
- Guidance on decision-making and procedures.

Such a framework would be capable of supporting flood defence specific decisions, which include maintenance and monitoring, as well as other flood risk management decisions, such as urban drainage, flood warning and development control. The DSS would also be able to consider the broader issues of river catchment and coastal zone management to ensure a more complete consideration of system performance. This is to become increasingly important as the UK implements the EU Water Framework Directive (EU, 2000) and the European integrated coastal zone management strategy (Commission of the European Communities, 1999).

Clearly a major hurdle in the implementation of a DSS will be related to the design of a suitable distributed computing system. However, there are other more complex issues relating to the design and integration of individual models. The broad range of decisions that need to be supported will require a suitably broad selection of modelling tools. For example, long term morphological evolution will not be considered on the same scale as short term flooding processes. However, the proneness to flooding of a coastal system is influenced by the shoreline morphology. Catchment scale modelling of fluvial processes can be achieved at a relatively coarse resolution, but may need to incorporate information from much higher resolution urban drainage models. Model integration and modularisation issues are the subject of ongoing research (*eg*. Gijsbers *et al*., 2002, Harvey *et al*., 2002 and Khatibi *et al*., 2003).

7.2.2. Refining proposed methodologies

Quantitative Risk Assessment

An obvious improvement to the high level methodology is the incorporation of a national DEM (Li and Baker, 2002) and CEH FlowGrid (stage-discharge curves at regular intervals along watercourses) when they are made available in the near future. This will significantly improve the quality of the high level assessment, as flood depths will no longer be based on a statistical analysis of a number of simulations for a given valley type.

The fragility curves used in the high level of the risk assessment methodology described in Chapter 4 are constructed with only limited evidence and derived from quantified analysis of documented failure mechanisms for a small number of defence types, back analysis of recorded failures and considerable expert judgement. Whilst defence specific verification of fragility curves can be provided by more detailed risk assessments, cost prohibits this for large numbers of defences. However, a more detailed analysis from these assessments may be used to validate the high level fragility curves. This may be achieved through direct comparison of a number of reliability-based fragility curves for representative defence structures and their high level counterparts. Quantified analysis of this type for a wide range of defences is laborious but would add substantially to the credibility of the high level risk assessment methodology.

As discussed above, only limited information was available to support the construction of the fragility curves used to describe the proneness of defences to breaching. Data on flood events and defence performance is essential if this type of approach is to be improved upon at the high level (and indeed to support further understanding of failure modes at more detailed levels). It is therefore strongly recommended that a database of flood events is set up*. This database needs to record information relating to the loadings placed on the defence (water level, wave height, duration *etc.*), the response of the defence (deterioration, overtopping volumes, breach width and invert *etc.*) and any impacts (flood extents, depths and duration, evacuations, damages *etc.*). Previous databases recording flood event information (*eg*. the World Health Organisation supported EM-DAT http://www.cred.be/emdat/) have tended to focus on the headline figures such as total economic damage or number of casualties rather than information of use to engineers aiming to learn from the failure of the flood defence system. The importance of recording adequate information has been recognised by those involved in handling major flood events (Roe, 1993), and there is often limited information available on flood events in the relatively distant past (Mosby, 1938) that should be incorporated into a database. However, this information needs to be centralised and future flood events need to be recorded in a consistent manner. Perhaps more important is the recording of 'near-misses'. These are events where flooding does not quite occur, or perhaps the defence is only overtopped in limited places. Recording these events as well as the failures will assist future understanding of defence reliability.

The more detailed level methodologies of the QRA have yet to be implemented. The intermediate level methodology has been described fully. The most detailed QRA has only been outlined and therefore needs to be refined before it can be demonstrated. It is likely at this level of the methodology the level of detail required to implement this tier of the methodology may require site specific treatment. Therefore a suite of tools and techniques with appropriate guidance may be recommended rather than a specific methodology.

Defence fragility

This thesis has contributed towards improved characterisation of defence structural performance. However, our understanding of defence breaching and deterioration mechanism is not complete and the methodology presented in Chapter 5 can only consider failure mechanisms for which a limit state function exists. Further research also is required in the areas of compound failure (when an initiating mechanism may suddenly trigger another failure mode). A large amount of research has studied the propagation of breaches in dykes and embankments (*eg*. Visser, 1998, Hassan, 2002 and Hanson *et al.*, 2002). This work is being extended further through projects such as IMPACT (http://www.impact-project.net/). The emphasis on studying embankment and revetment failure reflects the importance of such structures in protecting floodplains, however, research into other defences such as vertical walls needs to be considered. More specifically, mechanisms such as sheet pile 'peeling' when one section of sheet pile failing can result in the peeling away of a long stretch of other sections may benefit from further study. Study of the strength of old defences (*eg*. the Victorian flood walls in Cardiff) requires analysis if we are to have faith in the safety of these structures.

Wave overtopping has been the subject of a number of previous research projects that resulted in the publication of the Overtopping Manual (HR Wallingford, 1999). The calculation of overtopping rates is an essential part of a QRA as overtopping provides a direct threat to life and property. Research into overtopping continues under the CLASH project (Bruce *et al.*, 2002). However, whilst much research has focused on predicting discharges, further research needs to be targeted towards quantifying the effect of overtopping rates on defence reliability. This will enable improved definition of limit state functions for use at the more detailed tiers of QRA, and an improved understanding of structural response to better support the expert judgements used to define fragility at the high and intermediate level of assessment.

Performance-based decision-support methodology

Implementation of the decision-support methodology described in Chapter 6 requires that a hierarchical representation of system processes is constructed. The relationship between the performance of a child and parent process is described by a necessity (a measure of the extent to which failure of the child-process contributes to failure of the parent process) and sufficiency (a

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• *Note*: Databases such as this will also contribute towards better definition of other parameters used in the high level methodology that were not defined by the author, such as the representative breach width or flood duration.

measure of the influence that a given child process has on the performance of its parent process) value. A further measure of dependency is used to represent the amount of evidence originating from a common source or being influenced by common processes. Whilst these measures are necessary to capture the non-linear behaviour between system processes, aspects of the method for propagation of uncertainty merit further attention in order to improve the uptake of the approach.

The case study model for Burton-upon-Trent shows that there are a large number of repeated processes (for example, there are numerous embankments and vertical walls). As shown in Chapter 4, flood defences can be categorised into a limited number of generic types. The values of dependency, necessity and sufficiency used in uncertainty propagation are likely to be similar for a given classification of defence. For example, the integrity of the front face will be more important to the performance of a gabion wall than that of a concrete wall. It can therefore be expected to have a higher necessity value. This approach was adhered to in the construction of the case study, but lack of knowledge required these values to be assigned by expert judgement alone. This is justifiable as the modeller can assign values that allow the influence of the sub-systems on the super system to be propagated in a manner they believe corresponds to the behaviour of the real system. These expert judgements are recorded and so therefore auditable and open to debate. However, empirical evidence supporting these judgements would provide more confidence in the model. Being able to assign a value for necessity, sufficiency or dependency for repetitive processes with some degree of confidence also provides a useful model construction aid. The difficulty in quantifying these values more precisely stems from a lack of available data describing different system states. For some aspects of the system, this is already being addressed through DEFRA/EA funded research. For example, research into river conveyance aims to study the influence of factors such as embankment condition and the frequency of mowing grass on the embankment on the river's ability to convey water (which is one measure of the performance of the embankment).

For simple systems, the sensitivity of the performance of the super-system to dependency, necessity and sufficiency parameters can be easily explored by the user. However, flood defence systems, such as the example addressed in the case study are usually large. Exploring the sensitivity of the performance of the parent process to these parameters is less intuitive for systems with a large number of child processes and hierarchical levels. The sheer volume of child processes makes exploratory analysis time consuming. The influence of these parameters for such large and complex system has to be explored more fully. This can be achieved through automated sensitivity testing. If large numbers of sub-systems are to be assigned generic values as described above then the compounded effects of this need to be quantified. It may be more appropriate to assign bounds to the values of sufficiency, necessity and dependency to account for the uncertainty associated with defining them.

7.2.3. Broader development

Recalling the Source-Pathway-Receptor (S-P-R) model introduced in Chapter 3 as a convenient means of exploring different aspects of flood management, the contributions of this thesis are considered in the context of current and recommended future research. The overall contribution of this thesis has been a broad one. Whilst specific research targeted improvements in addressing flood defence condition characterisation (*i.e.* related to describing the pathway), a QRA provides a broad overview of the flood defence system performance that incorporates all three aspects of the S-P-R model (including a defence condition characterisation). This is further enveloped by a performance-based management methodology that considers those areas of the S-P-R model that can not be incorporated in a QRA.

Researchers in all aspects of the S-P-R model are constantly striving to improve the level of understanding of physical processes and translate this into improved models. All the methodologies presented in this thesis are flexible enough to not be dependent on the use of specific models, but can be updated as our understanding of the system is increased. For example, further understanding of breach propagation through physical and numerical modelling may be incorporated into the QRA at a number of different tiers. This may take the form of time dependent equations at a detailed level or empirical relationships at the high level. However, the consequence of using models is that they are only abstractions of our understanding of reality and are therefore always subject to uncertainty. Further advances in the understanding of meteorological phenomena, catchment processes such as the rainfall-runoff relationship, river and floodplain flow processes, storm surges and waves will enable improved modelling of the loadings on our system. Some improvements to existing models can be achieved through more complete representation of processes in existing models. One such example is the interaction of subterranean gravel layers near rivers that can result in flooding from ponding at the point where the gravel layer surfaces. This can be some distance from the river itself which may not be overtopping.

In other situations, however, our understanding is limited by insufficient long term data. Projects such as the national wave recording network (WAVENET) will help by improving model validation and the estimation of future loading conditions. In all cases however, a performance framework allows the costs (in terms of economic and other resources required for data acquisition) to be balanced against the gains in terms of uncertainty reduction.

New technologies are enabling more sophisticated means of data gathering. Satellites are providing land elevation maps and wind or wave data (such as the LandSat and ERS projects http://www.mimas.ac.uk/spatial/). A useful advancement for coastal zone modellers is LADS (Lidar airborne depth system) which can map both above and below the waterline simultaneously providing a more efficient means of collecting and merging DEM and bathymetry datasets (Stumpf *et al.*, 2002). Of key importance to the flood defence manager is the position and crest level of the flood defences. Helicopter based surveying is currently being trialled in the UK and may provide a means of fast and accurate surveying of defence crest levels along long stretches of embankment (*eg*. Burgess, 2002). Improved data gathering initiatives are crucial if we are to have a reasonable degree of confidence in our analyses. Regular monitoring is also essential for measuring how well a system is performing. Remote sensing technologies, such as heli-surveying and satellites should be tested and fully exploited where they can be shown to provide information that improves the decision-makers understanding of the performance of the flood system.

An area of research of particular importance to the assessment of coastal flood risk is the prediction of coastal morphology. The understanding of morphology is both important for assessing long term risks and defence reliability. An understanding on short (days-years), medium (years-centuries) and longer term scales is necessary. One storm can be enough to result in the breaching of a defence, perhaps from enormous and rapid toe scour – and therefore needs to be considered in defence design and maintenance decisions, whereas, long term understanding is necessary for taking strategic policy decisions. Current modelling can produce reasonable results in the short term (*eg*. Kamphuis, 1991, Nairn and Southgate, 1993) and qualitative approaches have increased our understanding of long term processes (*eg*. Burgess *et al*., 2002). Recent advances have been made in quantitative medium term morphological prediction (eg. Hall *et al*., 2002), but it is this timescale over which predictions are least certain and yet is the timescale over which most management plans are made. If accurate long term predictions of flood risk are to be made then these models need to be improved. Research continues in this area; a number of projects funded through DEFRA/EA funded research and European projects such as EUROSION (Serra *et al*., 2003) help to further contribute to our understanding of these processes. The relative success of modelling in the short term may be a reflection of the timescales in which our systems have been studied in detail to generate the models. In order to create more accurate medium term coastline evolution models, large scale and long term detailed site investigations are required. This may help quantify some of the long term, non-linear, morphological processes that are not observable in short term investigations or desk studies of historical trends.

Increased understanding of the S-P-R aspects delivers improved performance-based management. Aside from producing an increasingly accurate QRA and condition characterisation through improved modelling and monitoring initiatives, our understanding of system behaviour and the ability to better predict it increases. The performance of systems and their sub-systems can be evaluated and continually updated. However, not all system processes are amenable to quantitative description, whilst others are not monitored sufficiently. Substantial advances in performance-based management can be made if measures for these processes are identified and monitored. Environmental and social indicators need to be further developed, and their likely interaction with fluvial and coastal management options better understood, if we are to successfully balance them with economic indicators when considering issues such as long term sustainability. Some of these issues are being addressed by AIDEnvironment (http://www.aidenvironment.org/) and Tyndall Centre (http://www.tyndall.ac.uk/) funded research.

The move towards the performance-based management envisaged in this thesis will not be achieved through technological advancement and data gathering initiatives alone. It needs to be accompanied by organisational change within the flood management industry, and changed behaviour and understanding on the part of politicians, the general public, developers and other decision stakeholders. Floodplain development may still continue out of necessity to provide housing despite recent flood events, but in this case developers and planners should consider mitigation measures such as flood resistant buildings. It is most likely this will have to be driven by the industry, but current research is already addressing some of these issues (*eg*. Naylor *et al*., 2003). However, the implementation of a performance-based decision-support system will act as a catalyst for further change by enabling the advantages of these changes to be explicitly demonstrated.

7.3. IMPACT OF THIS RESEARCH

Increasing recognition of the need for more strategic and long term planning has resulted in a large number of different planning tools (*eg*. SMPs, CHaMPs *etc*.). These plans may often support contradictory objectives and weight conflicting information differently due to the nature of the investigation and the varying stakeholders involved in the preparation of the plan. A vision of performance-based management for flood defence systems has been outlined. This performance-based approach is consistent with balancing these conflicts and identifying options that result in the greatest contribution to overall performance. Whilst, inevitably, there is additional research needed to improve the measurement of performance and in further understanding the dependability of performance evidence and connectivity of sub-systems, this thesis has successfully demonstrated the advantages of performance-based management. A number of key areas for further research, some related to specific aspects of the research presented in this thesis, others related to broader aspects of performance and risk management, were noted in the previous Section. However, the importance of much of the research described in this thesis has already been recognised by decision-makers in the UK. This has been demonstrated by:

- the implementation of the National Flood Risk Assessment (HR Wallingford, 2003, Appendix G) using the methodology described in Chapter 4 has provided a national indicator of flood defence performance,
- the implementation of the high level risk assessment methodology described in Chapter 4 in the Foresight Flood and coastal defence project (Evans, 2003) which aims to support long term (over the next 30-100 years) flood and coastal defence policy,
- the acceptance of the concept of fragility curves as a useful indicator of flood defence performance and as part of the probabilistic risk assessment process,

- the incorporation of the concepts of performance introduced in Chapter 6 into forthcoming DEFRA guidance on performance evaluation and the Environment Agency's new performance-based asset management system (which the author has been invited to contribute towards) that is currently being designed.

Thus, there is a realistic prospect of the vision of the performance-based approach to management that is promoted in this thesis being implemented in practice.

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Appendix A

Definition of acronyms

AAD	Average Annual Damage
AOD	Above Ordnance Datum
BAP	Biodiversity Action Plan
BCR	Benefit-Cost Ratio
CFMP	Catchment Flood Management Plan
CIRIA	Construction Industry Research and Information Association
CHaMP	Coastal Habitat Management Plan
CZMP	Coastal Zone Management Plan
CMAM	Condition Monitoring and Asset Management
DEFRA (formerly MAFF)	Department for Environment, Food and Rural Affairs
EPSRC	Engineering and Physical Sciences Research Council
EA	Environment Agency
EIA	Environmental Impact Assessment
EU	European Union
FCDPAG	Flood and Coastal Defence Project Appraisal Guidance
FDMM	Flood Defence Management Manual
FDMS	Flood Defence Management System (software supporting the FDMM procedures)
FORM	First Order Reliability Method
FOSM	First Order Second Moment
GIS	Geographical Information System
IDB	Internal Drainage Board
ICE	Institution of Civil Engineers
ILD	Index of Local Deprivation
MAFF (now DEFRA)	Ministry of Agriculture, Fisheries and Food
MHWS	Mean High Water Springs
MWL	Mean Water Level
NAO	National Audit Office
NFCDD	National Flood and Coastal Defence Database
NRA	National Rivers Authority (now the Environment Agency)
ODPM	Office of the Deputy Prime Minister
QRA	Quantitative Risk Assessment
RASP	Risk Assessment for Strategic Planning
SMP	Shoreline Management Plan
SoP	Standard of Protection
SORM	Second Order Reliability Method
SoS	Standard of Service
USACE	United States Army Corps of Engineers

Appendix B

Glossary of terms

Coastal cell	A frontage within which longshore and cross-shore transport of beach material takes place independently of that in adjacent frontages.
Coast protection	Measures to protect the land against erosion and encroachment by the sea.
Coastal defence	An overarching term that includes sea defence and coastal protection.
Critical ordinary watercourses	Ordinary watercourses which the Environment Agency and other operating authorities agree are critical because of their potential to put at risk from flooding large numbers of people and property.
Fluvial defence	Measures to help prevent flooding from rivers.
Flood defence	An overarching term that includes both coastal and fluvial defences.
Flood risk	Combination of the probability of flooding and the consequences (economic or otherwise) of flooding.
Flood risk assessment	Consideration of the risks inherent in flooding.
Fragility	The probability of failure conditional on a given load.
Hazard	A situation with the potential to cause harm.
Main rivers	Watercourses designated as such on main river maps and are generally the larger arterial watercourses.
Ordinary watercourses	Watercourses that are not designated as main river.
Probabilistic discounting	The use of probabilities and probability distributions in order to account for uncertainty when estimating present values of cost or benefit over a period of time.
Reach	A length of channel or coastline between defined boundaries.
Return period	The average length of time separating extreme flood events of a similar magnitude.
Risk	A combination of the likelihood and consequences of harm being realised from a hazard.
Risk management	The activity of mitigating and monitoring risks.
Sea defences	Measures to help prevent flooding from the sea.
SoP	The level of protection provided by a structure. For fluvial defences this corresponds to the expected flood event that will lead to overflow. For coastal defences this is the storm event that will lead to serious overtopping.
SoS	The degree to which the EA provides, or seeks to provide, service. Normally achieved through capital works, routine maintenance and the operation of control structures.
Uncertainty	A reflection of our lack of knowledge or sureness about something or someone, ranging from just short of complete sureness to an almost complete lack of conviction about an outcome.

Please also see Appendix F for a glossary of terms that are specific to the PERIMETA software and decision-support.

Appendix C

Official guidance on flood defence management

This Appendix provides a summary of the procedures within published guidance from the Environment Agency and the Department of the Environment, Food and Rural Affairs. The main document used by the Environment Agency is the Flood Defence Management Manual and the corresponding Flood Defence Management System (the supporting database). However, the National Flood and Coastal Defence Database has just superseded the FDMS and the FDMM procedures are currently being reviewed.

C.1. THE FLOOD DEFENCE MANAGEMENT MANUAL

With respect to river defence projects, small projects are appraised and prioritised according to the Environment Agency's Flood Defence Management Manual (FDMM). This acts as a guidance document for the procedures required by the Flood Defence Management System (FDMS). The FDMS acts as a database for the EA and has some of the FDMM's prioritisation and justification methods included as pre-programmed routines. Data from the FDMS has now been migrated into the EA's new National Flood and Coastal Defence Database (NFCDD) which does not include any in-built decision-support functionality. Asset management procedures within the FDMM are supported by the Condition Assessment Manual (Glennie *et al.* 1991) which is used as a guide to help flood defence inspectors assess defence condition. The present methodology for condition assessment is discussed in detail in Chapter 2 and a new methodology is proposed in Chapter 5.

The FDMM defines and uses the Standard of Service (SoS) which is "*the standards to which the EA seek to alleviate flooding and allow provision for adequate drainage of land*". It is therefore the target standard of protection the EA aim to provide. It is also used as an indicator of performance giving an indication of the actual flood risk at a given site. The SoS was developed "*as a management tool providing a means for the definition and monitoring of flood defence SoS on a consistent and objective basis*". The system provides an estimate of the appropriate level of protection but does not endeavour to optimise the level of defence.

The overview of the FDMM in Figure C.1 shows that there are two main streams of data input and three main decision outcomes. The principal items of information input into the system are as follows:

(1) A condition characterisation number which summarises the inspection information on the state of the existing asset, guidelines for which are laid out in the Condition Assessment Manual.

(2) Detailed surveys are required in order to evaluate the assets in the flood risk area *i.e.* the potential consequences of flooding.

(3) Data about previous floods and predictive models of flooding are gathered (although this can often be limited) so that estimations can be made of the likelihood of flooding.

(4) Cost estimates are required to compare alternative implementation and scheme options.

The decision outcomes are either:

(1) 'Do nothing'

(2) Improve present SoS (capital scheme)

(3) Perform structural repairs (periodic maintenance)

(4) Cleaning, mowing *etc.* (routine maintenance)

Routine maintenance is prioritised on the basis of the service provided (using the SoS number as a prioritisation indicator) and periodic maintenance is provided using the condition characterisation (asset management) approach. Capital projects, if appraised using the FDMM consider economic, environmental and social factors as well as the SoS and integrity of any defences.

C.1.1. Asset monitoring

The asset management process involves regular surveying and monitoring of flood defences. In line with DEFRA's high level targets (DEFRA, 1999) a fixed inspection frequency is being phased out in favour of a risk based approach. A condition assessment of the flood defence on a scale of 1 to 5 and a residual life output are the main outputs of asset monitoring. The condition characterisation methodology is described in more detail in Chapter 2 of the thesis. Any asset identified as being in poor condition should be considered for a maintenance or replacement project.

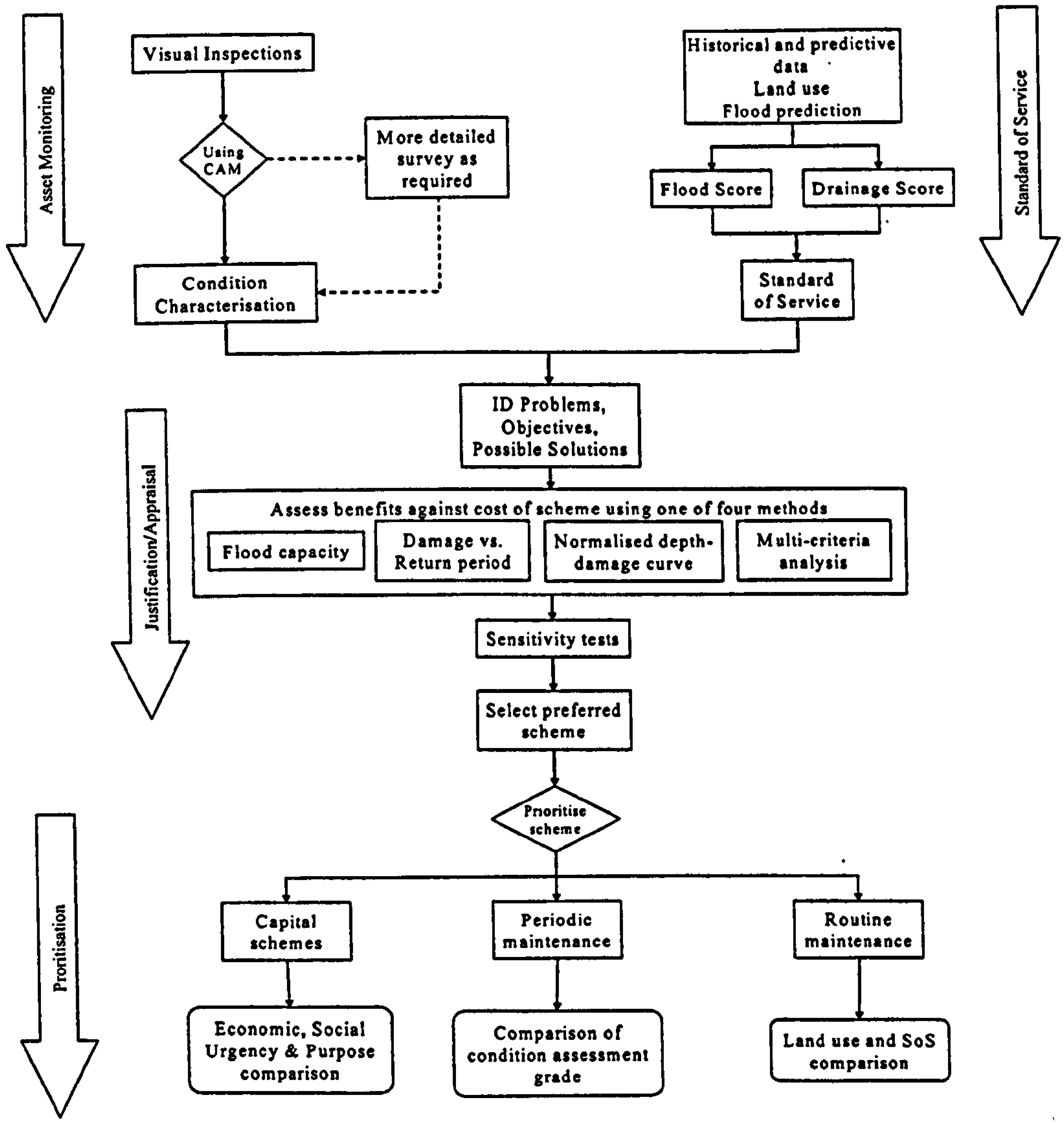

Figure C.1 An overview of the FDMM procedures

C.1.2. Calculating the Standard of Service

The SoS is a measure of expected annual damage expressed in terms of house equivalence which is defined as "*average financial cost of damage caused to an average house when flooded*". For example, a house will have an HE of 1.0, whereas an office has an HE of 0.033/m^2. The value of one House Equivalent is updated annually. The three steps to calculating the SoS are:

(1) Estimating the expected extent of flooding from historic and predictive modelling data.

(2) Estimating the damage due to flooding and inadequate drainage (measured in House Equivalents per km along the reach per year) from surveys of the assets in the flood risk area., and,

(3) Combining the estimates of probability and consequence of flooding to generate an expected annual damage. This is then compared with a target standard, derived from judgements of the appropriations of existing standards on a national basis.

Flood extent assessment

Benefits from a flood defence scheme are calculated from the potential damages in the area being considered for protection. This area is defined in the FDMM as the greater of the "*maximum known flooding extent*" or "*the area protected by an existing flood defence scheme*" and is referred to as the flood risk area. The flood extent is estimated using all available evidence, including maps, flood alleviation reports (giving location of schemes, defended areas and return periods of protection), incident reports, modelling studies and other surveys where appropriate.

The values of all farmland, residential and non-residential property within the floodplain are aggregated and averaged over the length of the river reach to give a value of HE/km which is used to classify the flood risk area into land usage bands used to set an indicative standard of protection. Land use bands are defined in Table C.1.

Table C.1Assigning a land use band based on potential damages

Land use band	Range of HE/km (flood and drainage total) for one river bank	Comment
A	>=50	Large urban areas
B	=25 - <50	Less extensive urban areas, high grade agricultural land
C	=5 - <25	Large areas of high grade land, some properties at risk
D	=1.25 - <5	Mixed agricultural land
E	>0 - <1.25	Low grade agricultural land
X	0	No known risk of flooding , no perceived drainage benefit

Note: There are two more bands Xld and Xest that represent watercourses which are perceived to act purely as a land drainage channel or are within an estuary and HE/km is defined as 0 for each

Calculating the SoS flood score

The flood score is an evaluation of the expected annual damage per kilometre from flooding, the drainage score is the damage resulting from inadequate drainage on farmland. Techniques using historical and predictive methods are used to calculate a score in terms of HE/km per year.

The historical technique averages the potential damages that would result from the previous floods where they to occur at the time of analysis. The predictive technique estimates damages using predicted flood extents for a range of return periods to give an expected average annual damage (AAD) for a reach. The average of the two flood scores should be used except when poor data limits the reliability of one of the outputs.

Calculating the SoS drainage score

As with the flood score, an AAD in terms of HE/km per year is calculated using both an historical and predictive technique. The historical technique relies on local knowledge and visual indicators to highlight where inadequate drainage has caused a reduction in crop production. The predictive technique identifies land with "poor" drainage by comparing the required freeboard and with the freeboard at times of dominant water level which is obtained from flow readings. Both the historical and predictive scores are averaged except in the case of a large discrepancy when further investigation should be undertaken.

SoS: Actual Standard of Service

The actual SoS is the sum of the flood score and the drainage score. This gives the total expected economic damages over the course of one year along one kilometre of a reach in terms HE/km per year.

SoS: Target Standard of Service

The target standard of service is the SoS that the EA has deemed to be an acceptable annual damage and is derived from the land use band. Typically this standard is between 0.5-1.0 HE/km per year. An actual SoS that is greater than 1.0HE/km per year implies the SoS is below target, requiring maintenance to be increased or a capital project to be undertaken.

C.1.3. Justification of Expenditure

Comparison of the actual and target SoS identifies reaches that may benefit from increased investment. Investments need to be justified and the FDMM describes four different economic justification approaches, however, only one is supported by the FDMS. Within the justification and appraisal methodology no use is made of the SoS values or the condition characterisation. The main input is the benefit assessment from the earlier surveys.

The supported method justifies investment using a benefit cost ratio (BCR). Work with a BCR>1 can be justified, obviously a higher value represents a more economically beneficial project. The use of the BCR is to allow an economic comparison between options, however the FDMM recommends that other benefits should be considered even if they can not be quantified.

Two of the methods involve calculating the estimated damage for a given return period and using this as a benefit value. The first is based simply on land values and uses a normalised damage value curve – this method is recommended only for minor projects or those with little data. The other method calculates the benefits from increased channel capacity.

The fourth method is similar to the old MAFF project appraisal guidance (MAFF, 1993), it is more rigorous and used only for larger projects. This method bases damage estimation on flood depth as well as flood duration. Damages to property, agriculture, economic losses resulting from disruption to traffic and the cost of emergency relief are all calculated. Environmental and social considerations may also be considered.

C.1.4. Prioritisation of Expenditure

Limited funding requires a prioritisation method, this varies according to the type of project being undertaken.

Emergency response and system operation

Costs incurred for operating the flood defence system and responding to emergencies are given maximum priority and funds are made available as required.

Periodic maintenance

Periodic maintenance consists of structural repairs, equipment repairs, painting, embankment repairs and vermin control. This is prioritised using the condition grade from the Condition Assessment Manual. Assets with structural elements classified as condition 5 ("very poor") should be acted upon immediately.

Routine maintenance

Routine maintenance involves grass cutting, weed clearing, dredging and rubbish clearing to maintain the defence's level of performance. Prioritisation compares the actual SoS value and the land use band. Although there is no formal requirement or suggested methodology, the decision maker is advised to consider environmental, seasonal and agricultural impacts, as well as systems effects and work scheduling difficulties.

Capital projects

Prioritisation of capital projects is a little more demanding. The FDMM does provide a methodology for prioritising capital projects, but frequently the DEFRA guidance (DEFRA, 2000a, 2000b and 2000c and 2000a, 2000b) are used, partly because this is a necessary condition to obtain a DEFRA grant for the scheme, but also as these methods are more thorough and tested.

The approach involves using a number of criteria to give a combined priority rating. Four factors are considered: economic, social, urgency and purpose (*ESUP*). Other influencing factors that should be evaluated but not used in the original decision-making formula are environmental benefits, heritage benefits and potential implementation problems. Prioritisation procedures can be overridden if:

- there are legal requirements(such as Health & Safety),
- the works so minor (<£5000) analysis is not worthwhile, or,
- the works are essential because other work (*eg.* maintenance or another capital project) is prevented, resulting in increased flood risk, unless these are carried out.

The *economic analysis* ranks the discounted BCR of the scheme into six bands (*eg*. BCR>5 means very worthwhile economically so scores six out of six, BCR<0.5 means there must be a reason other than economics to justify the scheme as it receives no score).

The *Social rating* counts the number of houses that would benefit from the flood protection. The more houses that benefit from flood protection the higher the social rating (for example if more than 1000 houses were to be protected the social rating would score the maximum six points).

The *urgency rating* is calculated by using one of two methods. If the project involves asset renewal or replacement then the highest score from both methods outlined below should be used, if this is not the case then only the first method should be used.

Method 1 is based upon the difference between the pre- and post- project Standard of Protection (SoP). The greater the difference between the two standards, the higher the urgency score (up to a maximum of 6).

Method 2 uses the residual asset life which is based upon subjective engineering assessment and knowledge of its history. A lower residual life receives a higher urgency score. A residual life assessment is often based on a correlation with the condition characterisation.

The *purpose rating* gives extra points if the area to be defended includes existing 'customers' (a customer is a property or landholder who has assets within the flood risk area), or if the defence protects urban land, or if its a coastal defence. This therefore represents a policy weighting.

These factors are combined and weighted; these weightings are set at a national level.

$$\text{Combined ESUP rating} = [(W_1 \times d_1)^4 + (W_2 \times d_2)^4 + (W_3 \times d_3)^4 + (W_4 \times d_4)^4]^{0.25} \qquad \text{(C.1)}$$

where $d = \{1\text{-}(\textit{Actual Ranking/Maximum ranking})\}$ and W is the weighting.

This *ESUP* rating is then placed into the following formula to from a *UTIL* rating in order to compare project priorities in the long term:

$$\text{UTIL} = 392 - (1000 \times \text{Combined ESUP rating}) \qquad \text{(C.2)}$$

A higher *UTIL* rating implies a higher project priority.

Flood warning

Flood warning usually receives a higher priority than maintenance and capital projects because of importance in minimising loss of life and also mitigating economic damages. If financial justification of flood warning is performed using the FDMM methodology, then it is done on the basis that for the first 4 hours flood warning, there is a 10% reduction in damage per hour advance flood notice. More detailed studies of the benefits of flood warning have been undertaken by the Middlesex University Flood Hazard Research Centre (Penning-Rowsell and Chatterton, 1977 and Middlesex University, 2002).

C.2. SHORELINE MANAGEMENT PLANS

Shoreline management plans (SMPs) (DEFRA, 2001c) provide guidance on taking a strategic approach to coastal defence over a 50 year period. The guidance does not specify technical methods, suggesting the engineer makes use of the most suitable and up to date analysis techniques.

SMPs are prepared by coastal groups which include the EA and Maritime Local Authorities. SMPs set out a strategy for sustainable coastal defence within coastal sediment cells, taking account of natural coastal processes and human and other environmental influences and needs. A SMP sets objectives for the future management of the shoreline based on predictions of the likely future evolution of the coast and knowledge of coastal processes within the cell. The methodology involves assessment of a range of strategic coastal defence options and identification of a preferred approach for sections of coast within the plan area. The preferred option is selected after consideration of the risk to people and the developed, historic and natural environment. The generic decisions are:

- do nothing,
- hold the existing defence line by maintaining or changing the standard of protection,
- advance the existing defence line, or,
- retreat the existing defence line (managed retreat or realignment).

The plan should consider all stakeholders of the coastal zone, and much of the planning process involves consultation with the general public. The plan should comment on the key shoreline issues showing how the shoreline is expected to evolve. The preferred planning options should be fully justified using economic (DEFRA, 2000a) and environmental appraisal (DEFRA, 2000c) guidance. An action plan of their implementation should be prepared (including a monitoring programme). The plan should identify the risks and uncertainties and in particular the implications of climate change and sea level rise on long term management.

The choice of a preferred option is used to inform future planning decisions relating to the coastal flood plain. Development plan policies, flood and coastal defence strategy plans and decisions therefore all need to take account of SMPs. Where the preferred option is either non-intervention or retreat, planning policies should strongly discourage further development in low-lying areas behind present shorelines. Additional development in such areas could unnecessarily commit flood defence authorities to expensive and unsustainable policies, which may in turn adversely affect biodiversity or other areas of the coast.

C.3. PROJECT APPRAISAL GUIDANCE SERIES

Whilst smaller schemes and day to day operations and maintenance are regulated using procedures laid out in the FDMM, larger projects are frequently regulated by DEFRA. To qualify for grant aid from DEFRA, ranging from 15% to 85% of the total project costs (Venables, 1998), the scheme must be appraised according to DEFRA's Flood and Coastal Defence Project Appraisal Guidance (FCDPAG) which supersede the previous guidance (MAFF, 1993) which was much less detailed and focused predominantly on economic appraisal. There are six documents in the series, FCDPAG1 (DEFRA, 2001a) provides guidance on combining the different aspects covered in volumes 2–6 and recommends how to use the document series and how technical issues, climate change and sustainability may be incorporated into the project appraisal process. In recognition of the speed of advancement of technical appraisal methods, as with SMPs, the guidance notes focus on the approach that should be taken rather than explicitly defining technical standards and methods.

C.3.1. The Strategic Approach

FCDPAG2 (DEFRA, 2001b) sets out a framework for strategic planning for areas at risk of flooding or erosion. Until relatively recently flood defence works have been appraised and implemented on an individual basis, frequently resulting in undesirable or unforeseen effects in other areas of the coastal cell or river catchment. As well as reducing the risk of adverse effects, taking a strategic approach offers the opportunity to make savings by more efficient allocation of resources. Strategy plans should identify appropriate solutions to meet the aims and objectives of SMPs and CFMPs.

Identifying problems and key issues

The first step in making a strategy plan is to identify the key issues, these may have already been identified in larger-scale plans such as SMPs. The strategy plan needs to consider an area that ensures all major process and impacts of the strategy are captured within the area considered. As with SMPs the strategy plan should consider change over a 50 year period.

Developing the strategy

Strategic objectives are developed by consultation with all stakeholders. Options that meet these objectives are identified and investigated. The 'do nothing' option is always analysed and considered as a baseline for comparison. The options are appraised on the basis of environmental (FCDPAG3), economic (FCDPAG5) and technical aspects. Issues of risk and uncertainty are discussed in FCDPAG4 (DEFRA, 2000b) which also describes methods to aid decision-making and option selection. The final strategy plan provides an outline over a 50 year period of how to meet river and coastal defence objectives for the area, providing a guide for how these objectives will be met by specific schemes.

C.3.2. Economic Appraisal

FCDPAG3 (DEFRA, 2000a) identifies methods for valuing costs and impacts in monetary terms and also sets out a recommended decision process, based on economic values. Guidance is provided as to how to make a benefit-cost analysis for flood defence projects. The economic appraisal guidance is designed to facilitate integration into a risk framework.

Evaluating the benefits

The first stage in the evaluation of the economic gains from a scheme is to identify the potential losses within the system boundaries. These are then used to assess the potential benefits from:

- flood alleviation,
- coastal protection,
- the effects of climate change,
- pumped drainage, and,
- flood warning systems.

Guidance is provided for assessing actual economic values.

There are several sources of data suggested for assessing economic damages. These include data from actual events, assessments from loss adjustors, adjusted council tax bands and standard data sets such as FLAIR (Middlesex University, 1990) and the Red Manual (Parker *et al.*, 1987) which will soon be superseded by the Multi-Coloured Manual (Middlesex University, 2002). Special consideration is given to temporary or semi-permanent structures and infrastructure value (including road, rail, pipes and cables). Future development is not to be considered when assessing potential losses. Agricultural losses are considered in terms of changes in net agricultural product, with special considerations necessary for land subject to quota agreement.

Indirect losses such as a decrease in trade, or disruption of road and rail networks are based on standard datasets in The Red Manual (Parker *et al.*, 1987) and COBA (DOT, 2001). Intangible losses, such as increased stress or damage to health should be considered if these impacts are thought to be unusually high. Environmental appraisal guidance is provided in FCDPAG5 (DEFRA, 2000c).

In the UK, flood damages are dominated by the depth of water and the volume of sediment, debris and sewage. Coastal flooding causes more damage than fluvial flooding due to the high salinity content of the water.

The benefits from flood alleviation schemes are calculated as the difference between the expected value of damage from a scheme and the expected damages from the 'do nothing' option. This is discounted over the life of the scheme to give a present value of the benefits. It may be necessary to estimate breach probabilities, these are then adjusted over the life of the scheme to reflect maintenance intervention and deterioration. FCDPAG4 provides a little guidance on how to estimate and use these failure probabilities, but in England and Wales this is dominated by judgement. If losses (for example floodplain development) or loadings (for example as a result of climate change) are expected to change these may also be adjusted over the appropriate time in the scheme's life.

The benefits from coastal erosion schemes are the difference in losses between the option and the 'do nothing' scenario. A probabilistic approach to erosion is used, these probabilities are estimated based on judgement and where available modelling and past behaviour of the cliffs.

Climate change is accounted for by assuming a 4.5mm/year rise in sea levels for the next 50 years, this should be added to a regional rate to account for land movements.

New pumped drainage schemes are unlikely (DEFRA, 2000a), but old pumps may require replacement. The dominant factor is the failure probability of the pump, however, newer pumps may also bring benefits such as reduced maintenance or running costs. Again, these failure probabilities are estimated by experts.

The benefits from flood warning are calculated by assessing the decrease in losses resulting from the improved warning system. A key benefit of flood warning is the reduction in loss of life.

The decision-making process

The main tool used to justify schemes is the benefit-cost ratio (BCR) using the Net Present Value over the scheme's life. The preferred choice is the highest BCR within the indicative Standard of Protection (SoP) needed from the defence, as specified by DEFRA guidelines. However, other factors will influence this decision, such as environmental considerations, uncertainty regarding the economic outcomes, planning constraints and the lack of funds. The BCR may be predicted over a long time period to assess the economic sustainability of the option. Probabilistic discounting is used to represent the loss in performance of an asset over time (by increasing its failure probability) and provide decision-makers with a value of expected economic damage over the life of the scheme.

C.3.3. Approaches to Risk

FCDPAG4 (DEFRA, 2000b) encourages the proper consideration of risk issues in the derivation of appropriate economic values and decision making, as set out in FCDPAG3. The depth of analysis used for a risk assessment should reflect the scale and impacts of the project. Guidance is given for a number of qualitative and quantitative approaches to risk assessment and identifying and handling the uncertainty associated with flood defence management. Qualitative and quantitative methods to estimating failure probabilities are introduced and methods of estimating system failure probability (for example using fault trees) are described. The concepts of risk are central to this thesis and so a more thorough overview of risk assessment methods, including methods discussed in the DEFRA guidance has been given in Chapter 2. Methods used to estimate defence failure probabilities are tackled in Chapter 3.

C.3.4. Environmental Appraisal

Adoption of the strategic approach proposed by FCDPAG2 (DEFRA, 2001b) requires appraisal of non-economic indicators. FCDPAG5 (DEFRA, 2000c) provides guidance on consideration of environmental issues in flood and coastal defence decision-making.

Guidance

The Environment Act of 1995 specifies that the EA and IDBs must contribute to the conservation of nature and heritage when carrying out flood defence duties. MAFF (1996) published guidance on selecting a preferred environmental option which should be used to identify the flood defence scheme options most suitable for meeting environmental objectives. Suitable options are then taken forward for a detailed benefit-cost analysis.

Initially, an appraisal of environmental effects is made at a strategic level to enable combined scheme effects to be considered. An individual environmental appraisal is then required for all flood defence schemes, this frequently takes the form of an Environmental Impact Assessment (EIA) (DETR, 1999). Undertaking of the environmental appraisal requires collecting data and analysing the likely impact on environmental factors relevant to flood and coastal defence projects, the most important of which are flora, fauna, local population, cultural heritage, property, landscape and geological/geomorphological features. Consideration of other factors such as access, amenity and economics should also be made. Since the environmental impacts are likely to have a wider influence than the local area, it is important to consider effects to a broader area.

It is government policy to encourage sustainable projects, and flood defence works should take into account the interrelationships between neighbouring defences and other processes within a river catchment or coastal cell. A long term approach, complimenting that of the SMPs and strategy plans should be taken. A key environmental indicator is biodiversity, and net gains or losses in biodiversity should be measured when implementing flood defence works.

Environmental evaluation

Not all environmental impacts can be measured in monetary terms and rigorous evaluation can be time consuming. The degree of implementation of such methods should reflect the scale of the project and the significance of the impacts. There are several methods for evaluating costs, the recommended method is a 'replacement' cost which reflects the cost of replacing or relocating a feature. This can be used as an economic environmental indicator, however other methods such as calculating an 'existence' value that reflects the perceived value to present and future generations may be more appropriate. Non-monetary indicators of environmental value include monitoring of populations or species.

A complete environmental evaluation will consider losses or gains (economic or otherwise) resulting from an analysis of nature conservation assets (*eg.* SSSIs and RAMSAR sites), changes in water levels (this links in directly with the WLMPs), losses in archaeological and heritage sites and impacts on the landscape. Clearly there is a lot of uncertainty with predicting environmental impacts and a lot of judgement is required in choosing appropriate assessment methods and evaluating impacts.

Habitat replacement costing

Habitat replacement costs may be required as a proxy for the value of lost habitats as part of an environmental evaluation, and, where habitat replacement is necessary to protect an important site that cannot be protected directly. The principal three costs of habitat replacement are land acquisition, land preparation and monitoring (pre and post project). These costs need to be discounted as in FCDPAG3.

C.3.5. Post Project Evaluation

FCDPAG6 provides guidance on undertaking post project evaluation. The purpose of this part of the guidance series is to review the work undertaken, report on the level of success and feedback appropriate lessons for future projects. The guidance will focus on using the concept of performance to measure success. The guidance will recommend that indicators of performance are measured and placed within a risk-based framework. The complete process will involve auditing the success of schemes, in terms of achieving economic aims and whether process objectives have been satisfied. Feedback of information into ongoing management of the scheme and external professional practice ensures that lessons learned can be used to improve best practice in engineering and management.

C.3.6. DEFRA Grant Aid and prioritising works

As it is unlikely that there will ever be sufficient funding for all flood defence works, a prioritisation score is calculated in order to target the most suitable schemes. The score is calculated based on a consideration of economics, social and environmental indicators. There is a maximum of 20 points for economic benefits, compared to a maximum of 12 points available for both environmental and social benefits. The additional weighting for economics ensures that schemes protecting industry, commerce and major infrastructure will gain appropriate priority when compared to housing.

The economics score is calculated using the BCR. This is the present value benefits divided by the present value costs as calculated using FCDPAG3 (DEFRA, 2000a). The score is calculated using Table C.2.

Table C.2 Derivation of the economics score for DEFRA grant aid prioritisation

BCR	Score
< 1	0
1 to 10.5	1 to 20 (linear scale)
> 10.5	20

The social score is based on the number of people that will benefit from the scheme, a hazard score and a measure of social deprivation. The score for the number of people is calculated using Equation C.3, with a maximum score of 8 being possible.

$75 \times$ *no. of residential properties / cost of scheme* (£k) (C.3)

Where the purpose of the scheme is to alleviate flooding, a hazard score of 1 is given if water levels would be expected to rise so fast that practical warning times would be less than two hours. A score of 2 is given if there is little opportunity to give an effective warning, or flood depths exceeding two metres or high flows are likely to be experienced. Other schemes receive a score of 0.

The social deprivation measure is based on the Index of Local Deprivation (ILD) (DTLR, 2001). This index ranks 8414 administrative wards in England and Wales using 33 indicators of six 'domains of deprivation'. These domains are: low income, employment deprivation, poor health and disability, poor education and training, poor housing, and poor geographical access to services. The 300 most deprived wards receive a score of 2, the next 1200 receive a score of 1. The least deprived 300 wards receive a score of -2, the next 1200 least deprived wards receive a score of -1. The remaining wards receive a score of 0.

The environment score is based on the area and importance of the habitat within the area affected. The scores are calculated by summing the following three equations.

$25 \times$ *area* (ha) *SSSI protected / cost of scheme* (£k) $\times$ 1.5 (C.4)

$25 \times$ *area* (ha) *other designated sites protected / cost of scheme* (£k) $\times$ 1.0 (C.5)

$25 \times$ *net gain* (ha) *of habitat that achieves BAP targets / cost of scheme* (£k) $\times$ 2.0 (C.6)

Heritage sites also contribute towards the environment score, a Grade 1 or 2* or scheduled monument receives a score of 2 and a Grade 2 monument receives a score of 1 providing a total score of 12 is not exceeded.

The sum of the economic, social and environmental scores is calculated and used to prioritise works. Those with a higher overall score are more likely to receive funding (although schemes with otherwise low scores can still receive funding to meet legal requirements).

C.4. OTHER GUIDANCE, CODES AND ORGANISATIONS

Many codes, manuals and organisation have relevance within the field of coastal and river engineering.

C.4.1. Guidance documents impacting flood and coastal defence management

Since about 1990, there have been a number of initiatives in regard to non-statutory plans that deal in particular with coastal issues. Many of these contain policies and proposals that have land-use planning implications, some directly involve flood defence managers, whilst others may impact only indirectly.

The *Planning Policy Guidance Note 25* (PPG25) (ODPM, 2001) is aimed at providing guidance to local planning authorities in order that they use their existing powers to guide, guide, regulate and control development in accordance with government guidelines. PPG25 aims to raise awareness to local authorities about the issues involved with development in flood risk areas and ensure that as well as local issues, catchment or coastal cell scale issues are also addressed. *Estuary management plans* (EMP) focus on ensuring a sustainable use of estuaries and are prepared by all major stakeholders. *Harbour management plans* (HMP) are similarly produced for harbours. *Coastal habitat management plans* (CHaMP) aim to develop sustainable coastal defence strategies in areas of important wildlife. *Local Environment Agency Plans* (LEAP) are produced by the Environment Agency on a catchment basis to develop a more holistic long term approach to achieving all its aims with respect to flood defence and other issues such as water quality, fisheries and recreation. *Water Level Management Plans* (WLMP) identify the water level requirements for a range of activities such as flood defence, agriculture and conservation. *River basin management plans* are required by the EU Water Framework directive (EU, 2000) which sets out the objectives of the water bodies in the river basin and how they will be achieved. Other plans that may have an impact on flood defence management are *Community strategies, Heritage Coast Management Plans, Biodiversity Action Plans* (BAP), *Integrated Coastal Zone Management Plans* (ICZM).

C.4.2. British Standards

There are many codes of practice that are relevant to coastal and river engineers, however use of these codes is not mandatory, neither does their use result in immunity against prosecution and they are usually used more as guidance rather than as a legal requirement (Fowler and Allsop, 2000). The most useful codes for coastal engineers are:

- BS6349 - Code of Practice for Maritime Structures
- BS8002 - Code of practice for earth retaining structures
- BS812, BS5328 and BS8110 - Codes of practice for use and specification of concrete
- BS1377, BS5930, BS6031 - Codes of Practice for geotechnical testing and earthworks
- BS6906 - Methods for testing geotextiles
- BS8004 - Code of Practice for foundations
- BS5493 - Code of Practice for protecting structures against corrosion
- BS5268 - Structural use of timber

There are also European and International standards, these are also not mandatory and provide no legal immunity.

C.4.3. Industrial and Research Organisations

Organisations such as CIRIA (The UK Construction and Industry Research and Information Association) or CUR (The Dutch Centre for Civil Engineering Research and Codes) regularly publish manuals and texts that provide state of the art information on data collection, design, planning and construction. Important references for flood defence managers include Protection of River and Canal Banks (Hemphill and Bramley, 1989), The Manual on the use of Rock in Coastal and Shoreline Engineering (CIRIA/CUR 1991), Seawall Design (Thomas and Hall, 1992) and The Beach Management Manual (CIRIA, 1996). The Institution of Civil Engineers, who represent engineers and the engineering industry in the UK, recognises the important role of engineers in flood defence management, regularly hosts related conferences and has recently published a report on best practice in river management (ICE, 2001).

C.5. ONGOING AND FUTURE DEVELOPMENTS

The *National Flood and Coastal Defence Database* (NFCDD) has recently superseded the FDMS as the data storage facility for the EA. The next phases of the development of the NFCDD will see an increase in the amount and quality of information available for use in flood defence management (Linford *et al.*, 2002).

Catchment flood management plans (CFMP) will provide a vehicle for considering holistic approaches to flood management at a catchment scale. The interim guidance follows an approach similar to that used to develop SMPs (HR Wallingford *et al.*, 2001 and Halcrow, 2001).

The *Modelling and Decision-Support Framework* has been implemented to support the development of CFMPs by automating and supporting parts of the process. In particular, using hydraulic modelling to identify economic and social risks and enable options testing.

The EA is currently designing a *Performance-based Asset Management System* (PAMS) in conjunction with the sixth instalment of the Flood and Coastal Defence Project Appraisal Guidance series (FCDPAG6). Both of these will be heavily influenced by the performance-based methodology introduced by the CMAM project.

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Appendix D

Review of flood defence failure

D.1. OVERVIEW

This Appendix provides a review of flood defence failure. The flood defences and erosion control structures that are considered are:

- Breakwaters
- Seawalls
- Dykes, levées and embankments
- Retaining structures
- Groynes
- Soft defences (beach and river management)

Control structures such as weirs, culverts and other mechanical assets are not reviewed. For each structure a summary of failure modes, limit state functions and design considerations is given. Because of the enormous number of physical processes and uncertainties there is often more than one limit state formula for each failure mechanism. Not all can be listed here, but the more commonly used limit state functions have been identified. This review is designed to compliment the author's study of flood defence failure mechanisms, it is in no way a design guide, nor does it reveal much of the detail behind important aspects such as estimating boundary conditions (eg. statistical estimation of joint wave and water level probabilities). For a more detailed information on coastal and river flood defence design the reader is referred to the references in this review and the main thesis.

D.2. BREAKWATERS

D.2.1. Brief summary

Breakwaters are often used to provide calm waters for a port as well as provide protection for a stretch of coastline. Shoreline breakwaters are constructed to protect the coast from erosion or alleviate flooding by the sea (or a combination of the two). This protection is provided either directly by providing a physical barrier or by protecting or encouraging the generation of a beach. Breakwaters take on many different forms and can be either shore-connected or detached. This is normally dependent on the functional requirements of the breakwater; a breakwater serving to protect land from erosion will often be detached (except when there are strong longshore currents). Port and harbour breakwaters have been comprehensively studied but this work is also of great value in the context of shoreline breakwaters, dykes and revetments.

D.2.2. Potential failure mechanisms of sloping face structures

The main failure modes of sloping face breakwaters are shown in Figure D.1 and described in the following sections.

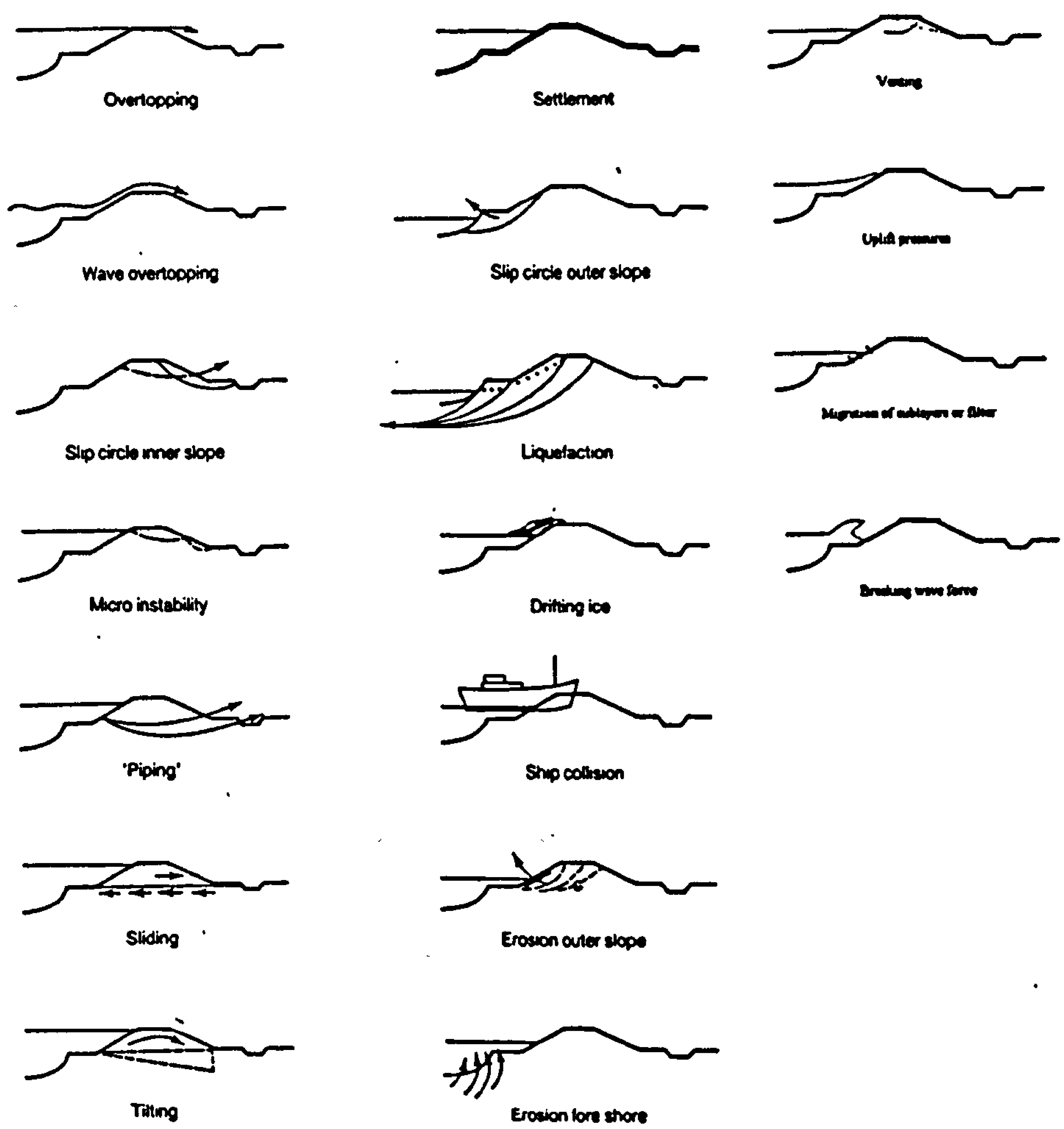


Figure D.1 Failure mechanisms of a sloping face structure (from CIRIA AND CUR, 1991 except venting and filter migration which are derived from Pilarczyk & Zeidler, 1996 and wave breaking force and uplift pressure which are derived from McConnell, 1998)

Overflow

The height of the breakwater is exceeded by the water level, this can result in inundation behind the defence and also erosion or damage of the crest.

Wave overtopping

The height of the breakwater is exceeded by the oncoming waves, the volume of water that overtops is often too small to be of concern, but if the volume is substantial it can result in lee side erosion.

Slip circle (inner slope)

This may be caused by a high phreatic level in the structure, this can occur as a result of a long period of high water.

Slip circle (outer slope)

This may be caused by a period of low water following a period of high water as the body of the structure may become saturated and collapse or slide.

Piping

An hydraulic gradient causes internal channels to form within the structure causing material to be transported along them and out of the structure.

Sliding

If the weight of the structure and its internal friction and cohesion are not enough to resist horizontal loadings (such as earthquakes or wave shock loadings) or changes in pore water pressure (caused by wave height and period) the structure or parts of it may start to slide.

Settlement and Tilting

The weight causes the subsoil to be compacted. As a consequence the crest level can be lowered (meaning there is an increased risk of overtopping). Differential settlements can result in tilting of the structure and uneven surfaces allowing rocks to be washed away more easily.

Micro instability

Micro instability of the structure's inner slope may occur because of seepage and a high phreatic plane.

Liquefaction

This occurs when pore pressures are so high that inter-granular contact is lost, meaning that the medium loses its shear strength. This occurs under cyclic loading from waves or earthquakes.

Drifting ice or ship collision

This can cause serious movement of the armour layer, or if large enough and impact is with sufficient force, the breakwater can be breached completely.

Erosion (outer slope)

This may be caused by wave attack (and also extreme ice and ship loads)..

Erosion (foreshore)

Waves and currents cause water movements on the seabed causing scour in front of the structure either at the toe or possibly at the berm.

Breaking wave force

This is most severe when the breaking waves are plunging, this can lead to armour movement or damage.

Venting

Wave induced water and pressure pushes infill material out (USACE, 2002).

Uplift pressure

Uplift pressures can be caused by an increase in phreatic level within the structure, this can be caused by either tidal or wave cycles. This difference in head may cause armour units or the filter layer to be moved.

Migration of sublayers or filters

Like piping, this is caused by hydraulic gradients created by waves or other water movements. This internal flow may result in the movement of fines from the inner layers to the outer layers or maybe even complete loss of material altogether resulting in settlement and loss of filter efficiency. Also if the filter gets blocked by migrating particles, it will become less efficient and more prone to other forms of failure.

D.2.3. General design

Overview

As with most other flood defence projects, there is a stage of data collection and analysis to obtain the design conditions. These are entered into the design formulae to provide dimensions for the breakwater and its components. Typical components and their respective functions of a breakwater are given in Table D.1.

Table D.1 Functions of breakwater components (CIRIA and CUR, 1991)

Element	Functions
Toe protection	Prevent erosion, increase geotechnical stability, stability of armour
Core	Attenuate wave energy, support armour, geotechnical stability
Berm	Attenuate wave energy, increase geotechnical and armour stability
Underlayer	Filtration, erosion protection of core, in-plane drainage, reduction of internal hydraulic gradients
Armour layer	Prevent erosion of armour layer by waves
Crest and crown	Attenuation of wave overtopping, access for maintenance and services

The design should be such that the components satisfy the required primary and secondary functions. Much of the general design is performed using 'rules of thumb' based around general experience, it is recommended to test all designs in a hydraulic physical model (CIRIA and CUR, 1991). Some of the more quantitative elements of the design are listed below.

Armour layer

The armour for breakwaters is designed using the Hudson formula or the Van der Meer formula which are also listed in Section D.5, the Hudson formula is used to specify the weight of rock required:

$$W = \frac{g\rho_r H^3}{K_D (s-1)^3 \cot\alpha} \tag{D.1}$$

where $s = \rho_r / \rho$ and K_D is chosen by the designer and influenced by many factors such as the whether the wave will break or not and the shape of the armour units.

The other possible method of armour size determination is that of Van Der Meer (1998) who derived formulae for armour layers of thickness $t=2*D_{n50}$ the formulae relate incident wave conditions, and the level of damage that may be allowed, to the dimensionless stability number, $H_s/\Delta D_{n50}$ for plunging waves:

$$H_s / \Delta D_{n50} = 6.2 P^{0.18} (S_d / \sqrt{N})^{0.2} \xi_m^{-0.5} \tag{D.2}$$

and surging waves:

$$H_s / \Delta D_{n50} = 1.0 P^{-0.13} (S_d / \sqrt{N})^{0.2} \sqrt{\cot\alpha}\, \xi_m^{P} \tag{D.3}$$

where P is a notional permeability factor (ranging from 0.1 for impermeable filter layers to 0.6 for non existent filter or core), S_d is the damage number which is defined as $A_e = D_{n50}^2$ where A_e is the erosion area, N is the number of waves and ξ is the Iribarren number which is defined $(H/L)^{-0.5}\tan\beta$.

Filter layer

The filter design should obey filter laws:

- $D_{15\,\text{filter}} < 5\, D_{85\,\text{base}}$ Stability or piping criterion
- $D_{15\,\text{filter}} > 5\, D_{15\,\text{base}}$ Permeability criterion
- $D_{50\,\text{filter}} < 25\, D_{50\,\text{base}}$ Uniformity criterion

Where D_n represents the n% value of the material's sieve curve.

Current attack:

This can often be predicted using the Shields formula (Shields, 1936) for critical shear stress, but this is only valid for unidirectional steady flow:

$$\psi_{cr} = \frac{\tau_{cr}}{(\rho_r - \rho_w)gD} \tag{D.4}$$

where ψ_{cr} is the dimensionless shear parameter (or Shields number), τ_{cr} is the critical shear stress at which the rocks or particles begin to move, ρ_r is the rock (or particle) density, ρ_w is the density of water, D is the grain size. Grain displacement for particles of above 5mm has been shown to occur at ψ_{cr}=0.03 and so this should be used as a conservative estimate in calculations. Many other formulae have been developed to aid in the understanding of complex flow. Due to the multitude of formulae involved this report will not list them (see CIRIA and CUR (1991) for more information). The key factors involved in combined current and wave attack are:

(1) Critical shear stress
(2) Particle size
(3) Fluid viscosity
(4) Bed roughness
(5) Structure slope

D.2.4. Specifics for structure type

The different types of breakwater structure are shown in Figure D.2.

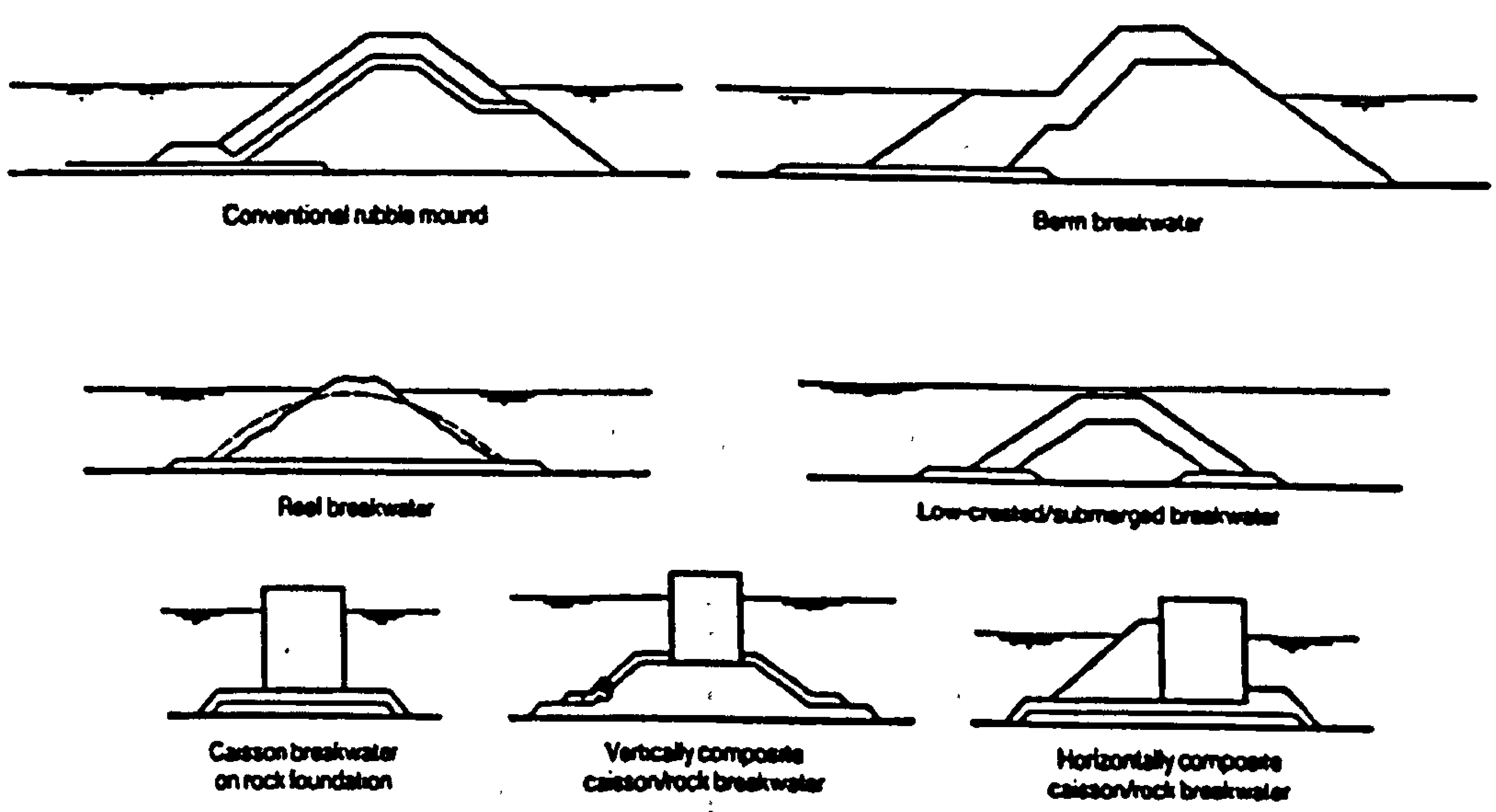


Figure D.2 The different types of breakwater (CIRIA and CUR, 1991)

Rubble-mound

Figure D.3 shows the principal failure modes of a rubble mound breakwater. This type of breakwater may also have a crown wall to allow for easier access or to enable the breakwater to perform other actions.

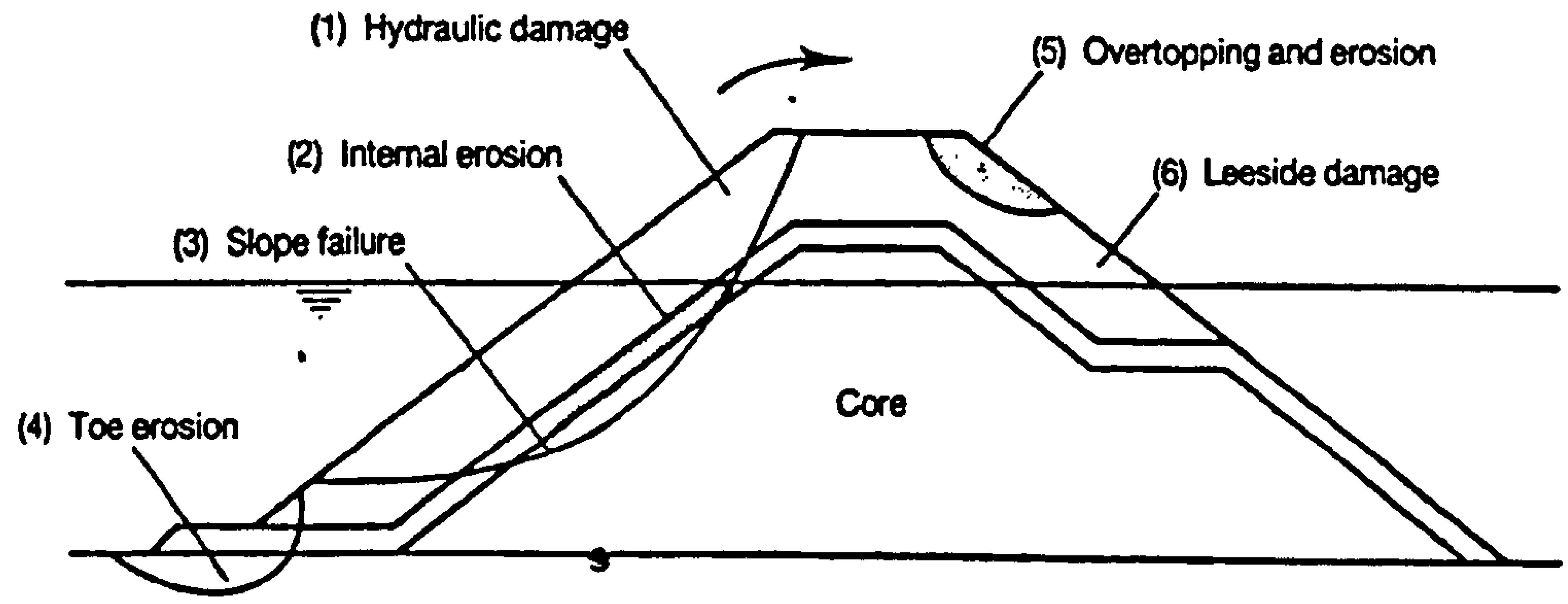


Figure D.3 Principle failure modes of a rubble mound breakwater (CIRIA and CUR, 1991)

If a crown wall is desired then its stability can be predicted either by modelling or using the formulae derived by Bradbury *et al.* (1988).

$$F_H = \left({aH_s}/{A_c} - b \right) * \left(\rho g h_f L_p \right) \tag{D.5}$$

where F_H gives the horizontal loading, L_p is the peak wavelength, h_f is the crown wall height, A_c is the armour crest level and a and b are empirical coefficients. The vertical force can be calculated as (assuming a triangular distribution of uplift pressure):

$$F_V = \left({aH_s}/{A_c} - b \right) * \left(\rho g B_c L_p \right) \tag{D.6}$$

where B_c is the crown width. Often a more conservative assumption of rectangular distribution is assumed which results in an uplift force of twice that given by the above equation being calculated.

Caisson on rock foundation

The key failure mechanism for this type of breakwater is toe erosion which is governed by the design storm.

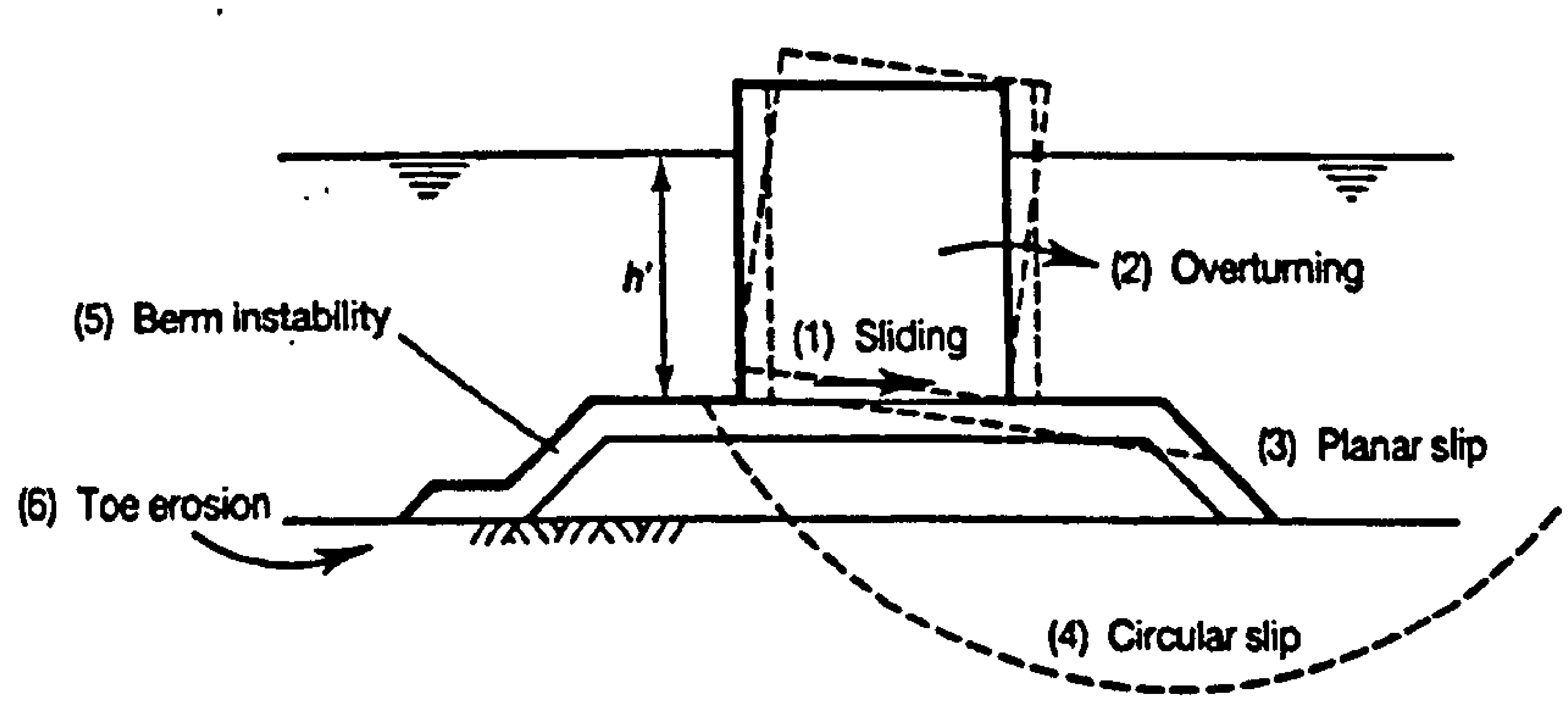


Figure D.4 Failure modes of a vertical breakwater on top of a rock foundation (CIRIA and CUR, 1991)

Horizontally composite rock breakwater

As shown in Figure D.5 the main difference between caisson breakwaters and horizontally composite breakwaters is the extra failure mode caused by the hydraulic instability of the mound.

This mound in front of the breakwater must break and absorb the wave energy effectively. The mound is usually concrete armour units that are highly porous. The hydraulic stability of this mound is similar to that of the primary layer of a conventional breakwater but with different coefficients that need to be obtained from model testing.

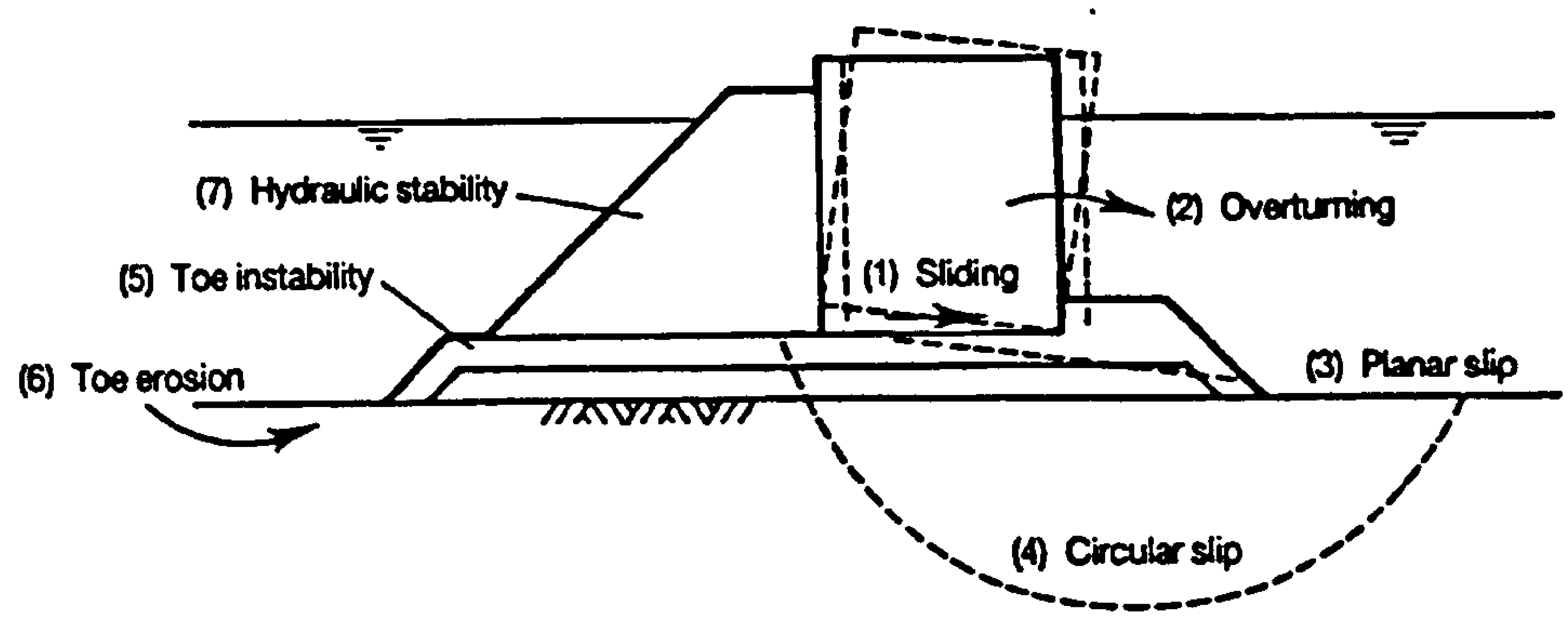


Figure D.5 Failure modes of a horizontally composite breakwater (CIRIA and CUR, 1991)

Vertically composite rock breakwater

This type of breakwater is better deployed in deeper waters. Waves are forces to break on the mound which can lead to high breaking forces on the caisson. It is therefore important to keep the mound low. The failure mechanisms are the same as for the caisson on the rock foundation, but the design of the berm may be different.

Berm breakwater

Berm breakwaters are designed to be dynamically stable, their slope can be either artificially engineered or allowed to be developed by hydrodynamic forces. Prediction of the optimum berm design can be done using a computer model such as BREAKWAT which is based on Van Der Meer's modelling tests (Pilarczyk and Zeidler, 1996).

Reef breakwater

These consist of a large volume of rock which can reshape to a dynamically stable profile under wave attack as shown in Figure D.2. The reduced crest height of a structure can be predicted according to Van Der Meer (1990) using:

$$h_c^* = \sqrt{A_t / e^{aN_s^*}} \qquad \text{(D.7)}$$

where a=-0.028+0.045C'+0.034(h_c'/h)-6.10$^{-9}B_n^2$

where A_t is the area of structure cross section, $C'=A_t/(h_c')^2$, h is the water depth at the toe, B_n is the bulk number (=A_t/D^2_{n50}) and N_s^* is a stability number.

Submerged breakwater

Similar to a rubble-mound breakwater but requires lee-side armour as wave energy is transmitted over the breakwater. The dimensions of this armour are determined by the rate of overtopping which can only be assessed by modelling. Other than the lee-side armour the design process is the same as for rubble-mounded breakwaters. The stability of a submerged breakwater is described by:

$$\frac{h_c'}{h} = (2.1 + 0.1S)e^{-0.14N_s^*} \tag{D.8}$$

where S is the damage level ($=A_e/D^2_{n50}$) where A_e is the erosion area.

D.3. SEA WALLS

D.3.1. Brief Summary

A sea wall is defined as a shoreline structure whose primary purpose is either protection against coastal erosion, alleviation of flooding or a combination of both. Sea walls are classified based on their shape and permeability. Sea walls can be vertical, sloped, or a combination of the two and can be either porous or non-porous and be constructed from many types of material.

D.3.2. Summary of failure mechanisms

Sea walls are prone to many causes of damage which can lead to one of the five main forms of failure of a sea wall as defined by Thomas and Hall (1992). These are:

(1) Flow through or under the wall
(2) Overtopping
(3) Damage of the wall front or crest
(4) Geotechnical instability
(5) Slope instability

Also stated in Thomas and Hall (1992) is the significant role of toe erosion (induced by low beach level) in the stability of the wall which can be a precursor to all five main methods of failure. CIRIA (1986) states that 12.3% of all sea walls surveyed have suffered from significant toe erosion which ranks it as the most common form of damage. Particular care should be taken when designing the toe so as to allow for future (possibly adverse) beach evolution.

D.3.3. General design

A prerequisite for determining design criteria is the selection of the overall design standards, *i.e.* the required life of the seawall and the acceptable risk of being overwhelmed by exceptional waves and/or tides (Thomas and Hall, 1992). Seawalls are designed with the consideration of cost, environmental impact (although often not in the past), structural and hydraulic performance, and their ability to fulfil their primary functions.

The design parameters are chosen based upon the data analysed, the desired design life and the risk of exceedance. The wall is designed so that it is stable with respect to the hydraulic and geotechnical loadings and the risk of exceeding design conditions is deemed acceptable.

D.3.4. Specifics for structure type

Sloping porous walls

These can be treated as porous revetments (such as rip-rap) which are covered in section D.5.

Sloping non-porous walls

These can be treated as non-porous revetments (such as concrete slabs) which are covered in Section D.5.

Vertical walls

These should be treated as vertical retaining structures (such as gravity walls or gabions) which are covered Section D.8. The structure should also be designed against wave pressure distributions such as Goda's (1974). The most significant influences being wave parameters, water heights and structure shape.

D.4. GROYNES

D.4.1. Brief summary

Groynes are long, narrow structures normally built perpendicular to the shoreline and sometimes have different shaped seaward heads (T, Y or round-head). Groynes are designed to interrupt longshore transport and cause a build up of sediment on the updrift side. Groynes can be constructed from many materials; including rock, timber, concrete, masonry and steel.

D.4.2. Specifics for structure type

The different types of groyne material, their usage and respective advantages and disadvantages are summarised in Table D.2. Usually, due to loss or movement of structural elements the performance of the groyne will deteriorate until a point it can be judged not to be able to fulfil its intended role. CIRIA (1996 and 1990) state that there are no fixed formulae or procedures for groyne design, the design process is based around experience and experimental results and differs for shingle and sand beaches.

D.4.3. Failure mechanisms

A groyne is adjudged to have failed when it can no longer perform its desired function. This will predominantly be due to loss or damage of structural elements (such as wooden planks or rocks). Although complete loss of a groyne due to local undermining or a combination of wave forces and loss of foundation support due to scour is not uncommon (National Rivers Authority and Binnie and Partners, 1993). Other potential sources of failure are vandalism and abrasion. Wood groynes are also susceptible to biological attack such as fungus and rot.

Table D.2 Groyne type, use, advantages and disadvantages (CIRIA, 1996)

Type/ Material	Advantages	Disadvantages	Suggested applications
Vertical timber	Possible post-construction adjustment	Cost and availability of hardwoods Environmental restrictions on hardwood sources Susceptible to physical abrasion and biological attack Vertical construction does not absorb wave or current energy Current induced beach scour pits along face and around head Unstable if large cross-groyne differentials in beach elevation develop or if large crest heights are required Difficult to construct below MLW Require maintenance	Low to moderate energy shingle beaches with low net drift
Rock mound	Hydraulic efficiency due to energy absorption Re-usable material Simple construction methods Underwater construction possible Post-construction adjustment easy Stable, durable No size limit	Availability and transport of suitable rock Structures may be hazardous to swimmers and other beach users Accumulation of debris within structure Bed layer required if substrate is mobile	Low to high energy sand or shingle beaches with low net drift in areas where suitable rock is available Good for terminal structures
Concrete units	Hydraulic efficiency due to energy absorption Stable, durable Availability of materials	Rigorous construction methods required May be hazardous to swimmers and scramblers Accumulation of debris within structure Bed layer required if substrate is mobile	Low to high energy sand or shingle beaches with low net drift, in place of rock Good for terminal structures
Vertical concrete/ masonry	Availability of materials	No post-construction adjustment Expensive and complex construction particularly below MLW Near vertical construction does not absorb wave or current energy Maintenance required	Low to moderate energy beaches with low net drift Good for terminal structures
Steel sheet piles	Rapid construction Can be placed below low water	Vertical construction Does not absorb energy No post-construction adjustment Suffer from abrasion; resulting jagged edges are a safety hazard Suffer from corrosion	Can be used to form foundation and sides of concrete structures, particularly below MLW
Gabions	Low cost, rapid construction Hydraulically efficient	Not durable Particularly susceptible to vandalism Only suitable for small structures	Low energy sand or shingle beaches with low net drift
Rock-filled crib work	Low cost due to smaller rock Hydraulically efficient	Movement of rocks can damage crib-work	Low to moderate energy sand or shingle beaches, with low net drift.
Grouted stone or open stone asphalt	Low cost Susceptible to abrasion	Prone to settlement problems	Low to moderate energy sand or shingle beaches, with low net drift, on stable substrate
Rock apron around timber	Increase energy absorption of existing vertical structures	Interfaces subject to abrasion due to different interactions with waves	Refurbishment of old vertical groynes on low to high energy shingle or sand beaches with low net drift

D.4.4. Design of groynes for shingle beaches

Little in the way of formulae are used to design groyne fields, guidelines have been given following research by Coates and Lowe (1993) and Coates (1994).

Material type

On shingle beaches, vertical and rock mound structures were found to perform equally up until H_s=2m when vertical structures start to produce upper beach erosion.

Crest level

The most effective structures were found to be high rock structures. The crest of groynes should be set at about 1m above the design storm beach profile. This design profile needs to reflect the influence of varying water levels and longshore sediment transport.

Groyne length

The landward end is taken as the beach head, or the area beyond which erosion is not acceptable. Research performed by Coates (1994) suggests that groynes are only effective to the point when they intersect the shoreline at a depth of $0.75H_b$.

Groyne spacing

This should be set so as to avoid the beach head not being exposed to any direct wave action within a groyne bay.

Groyne head extensions

These can be used to improve groyne efficiency. On shingle beaches scour at these heads is usually of little importance, meaning that they are not useful. If the lower beach is sandy, then the groyne extensions can mean reduced current dissipation and reduce sand scour, therefore increasing local stability. Large head extensions at an elevation at or above MHWS can modify the incident wave meaning that the groyne spacing can be increased.

D.4.5. Design of groynes for sand beaches

Groynes are most effective on beaches in micro-tidal low wave energy environments where the spacial distribution of sediment transport is limited (CIRIA, 1996).

Material type

On low wave energy beaches, vertical groynes perform as well as rock groynes, but for higher wave energy beaches rock is considered more effective.

Crest level

The higher the groyne crest (up to the beach berm level) the greater the effect on longshore transport.

Groyne length

The more of the surf zone covered by the groyne, the greater its efficiency. Designers are advised (CIRIA, 1996) to choose the breaker point of a moderate summer swell wave as the seaward extent of the groyne (although this could be longer if required).

Groyne spacing

Groyne spacing can be greater on sand beaches as they do not re-orientate themselves as quickly as shingle beaches.

D.5. DYKES AND REVETMENTS

D.5.1. Brief description

A revetment is a form of protection to protect against wave or current erosion. A typical revetment will include an armour layer, filter layer and possibly other sub layers and details such as those at the crest or toe). Dykes have failure modes similar to those of breakwaters and these are summarised in Figure D.1, the probable failure mode can depend on the revetment type. The main types of revetment are:

(1) Rock
(2) Gabion
(3) Block
(4) Bitumen
(5) Concrete
(6) Geotextile
(7) Others

As dykes are sea defence structures, their principal loadings are similar to those of breakwaters. However revetments can be used on river embankments, but the principal loadings and therefore probable failure mechanisms may vary.

There is much information on revetment design, and the main sources read by the author are of Dutch and British origin. These represent two different approaches to revetment design, as the Dutch approach is much more detailed and quantitative than the British methodology. In this report the British approach is covered completely and additional information used by the Dutch is also described.

D.5.2. Failure mechanisms

The key failure mechanisms for revetments are:

(1) Removal or damage of elements (armour stones, concrete blocks *etc.*) from the cover layer
(2) Uplift pressure resulting in armour layer damage
(3) Sliding of coverlayer
(4) Wave impact creating a percolation force
(5) Toe scour
(6) Overtopping
(7) Piping of material
(8) Gradual erosion
(9) Mass geotechnical failure of river bank

Gradual abrasion and corrosion should also be considered in design.

Revetments are generally not used to improve mass geotechnical instability of an embankment unless it is part of a composite structure (Hemphill and Bramley, 1989) and so the underlying embankment should be stable before designing the revetment.

Removal or damage of elements

This can be caused by a number of factors, but the most likely is wave energy, although extreme loads such as ice or ships may cause localised damage.

Uplift pressure

These can occur when the drop in groundwater level lags during a tidal cycle, this is most severe after a storm surge. Uplift pressures can also occur as a result of waves causing a change in water level at the revetment. As a wave runs up a permeable coverlayer water will seep inside, because the wave run-up length will be greater than the run-down, the phreatic level under the coverlayer is increased. This head difference may induce instability in the structure causing elements of the coverlayer to be forced out.

Sliding of coverlayer

This can occur when the frictional force between the slope and the revetment is exceeded or when toe support is inadequate.

Deformation of coverlayer

This can be caused by two processes, first the loss of fines from beneath the coverlayer (either from the filter or sub-layers) causes uneven settlement that leads to deformation of the coverlayer. Deformation can also be caused by wave attack leading to structural fatigue.

Wave impacts

Plunging breakers which break over a structure cause high, short lived but cyclic pressures. This can lead to brittle failure of rigid elements or deformation or fatigue of more flexible elements. Waves also create an impact force when they hit the revetment as well as up and down-rush forces before and after breaking.

Toe scour

This is caused by the presence of a structure and can result in undermining.

Overtopping

This can cause structural damage to embankments including lee scour. This can be reduced by causing a greater dissipation of energy by roughness or permeability.

Piping

This is caused by steep hydraulic gradients within the embankment causing soil to migrate through internal channels resulting in settlement. This can also lead to the blocking of the filter layer.

Gradual erosion

Caused by wave action shearing particles from the slope or wave action causing local increases in pore pressure that liquefy the soil.

Mass geotechnical failure

These modes are dealt with in detail in Section D.7.

D.5.3. General design

Revetment design involves similar considerations to seawall design in that previous data is used to provide design conditions. The revetment must be resistant to all forms of hydraulic loading (such as waves, turbulence and currents). Other factors such as access, environmental impact and cost should be considered. The type of revetment cover layer should be chosen after analysis of the local conditions and this will influence the design procedure and the likely failure modes. Other elements of the revetment are considered separately.

D.5.4. Structure specific design

Table D.3 provides a summary of the critical failure modes, the determinant loading and strength parameters for the most common types of revetment.

Table D.3 - Summary of the critical failure modes for each type of revetment (Pilarczyk, 1998)

Coverlayer	Critical failure mode	Determinant loading	Strength
Sand/gravel	Movement of material Loss of material	Velocity field in waves	Weight friction Dynamic 'stability'
Clay/grass	Erosion Deformation	Max. velocity Wave impact	Cohesion Grass-roots Quality of clay
Rip-rap	Movement of material Deformation	Max. velocity Seepage	Weight Friction Permeability of sublayer/core
Gabions/ mattresses and geotextiles	Movement of material Deformation Abrasion/corrosion of wires	Max. velocity Wave impact Climate Vandalism UV light	Weight Blocking Wires Large unit Permeability of rocks and sublayer
Placed blocks and block-mats	Lifting Bending Deformation Sliding	Overpressure Wave impact	Thickness, friction, interlocking Permeability of blocks and sublayers Cabling pins
Bituminous/asphalt systems	Erosion Deformation Lifting	Max. velocity Wave impact Overpressure	Mechanical strength Weight

Rock

Rock protection can be used under both heavy wave and current attack. The principal form of damage to rock revetments is loss or movement of stone units.

River revetment design

Some of the typical characteristics of rip-rap are summarised by Escarameia (1998):

- The rock layer is usually graded between D_{85}/D_{15} = 1.5 to 2.5
- The angle of repose should be between 35° and 42°.
- The angle of internal friction should be between 40° and 45°.
- The thickness should be not less than $2*D_{n50}$ or 1 to 1.5 maximum dimension
- Porosity should be between 25% and 40%

D_{n50} should be chosen as (Escarameia and May, 1992):

$$D_{n50} = C\frac{U_b^2}{2g(s-1)} \qquad \text{(D.9)}$$

where $D_{n50} = \left(\frac{W_{50}}{\rho_r}\right)^{1/3}$, and C is a coefficient that accounts for turbulence intensity, 0.1 should be used for continuous revetments and 1.25 for end revetments. It should be noted that there are other methodologies suggested for this, for example see Pilarczyk (1990) and Maynord (1993).

Coastal revetment design

For protection against wave attack, the Hudson formula (CERC, 1984) and Van Der Meer (1998) formula can be used. These formulae have already been defined in Section D.2.3.

Gabions

Gabions can withstand fairly heavy current attack and require less rock than a rip-rap revetment. However because they rely on steel to hold themselves together they can be corroded in saline environments. Abrasion attack is also significant in a sediment laden environment.

River revetment design

The most likely form of damage to gabions are abrasion, corrosion and vandalism. Corrosion can be relatively easily dealt with by coating or galvanising the wires. The rock size required is calculated by using Equation D.9, using C=12.3×TI-1.65 valid for TI≥0.12 where TI is the turbulence intensity. This equation is valid only for gabions of 300mm thickness, but it should be remembered that manufactures can give valuable information about their product's performance.

Mesh sizes are normally around 40-60mm, and are normally 2/3 of the average fill size. If placed vertically gabions are normally tilted backwards slightly to allow for future settlement.

Coastal revetment design

The gabion wires should be very resistant to corrosion if placed in a marine environment, however wave action is likely to move rocks within the gabion wires causing abrasion. Gabions are therefore seldom used under significant wave exposure.

Pre-cast Blocks

Loose blocks for river revetments

These are most suitable for channels where access by machines is difficult. They are also useful for difficult contours in the river (such as bends). The blocks can come in many sizes, materials and shapes and many variables are not fully researched.

Again using Equation D.9, the size of block to be used can be determined, but C=0.725 for continuous protection and C=0.95 for end protection.

Blocks should also be tested against sliding, the maximum frictional force is given by:

$$F_f = f \times [l_2 \times b \times t_a \times (\rho_c - \rho_w) \times g \times \cos\alpha \qquad \text{(D.10)}$$

where l_2 is the length of revetment under the water line, b is the block width, t_a is the block thickness, ρ_c, ρ_w are densities of concrete and water, α is the slope angle. The driving forces on the zone of wave attack will be:

$$F_a = [l_1 \times b \times t_a \times (\rho_c - \rho_w) \times g \times \cos\alpha] \qquad \text{(D.11)}$$

where l_l is the length of revetment under wave attack (=H_s/sinα). Overall stability is calculated by comparing the appropriate ratio of F_f/F_a.

Additional requirements for coastal revetments

A complex method suggested by Klein Bretler and Bezuijen (1991) has been simplified to:

$$H_s / \Delta t_a = S_b \xi_b^{-0.67} \qquad \text{(D.12)}$$

where t_a is the thickness, S_b is an empirical coefficient and $\xi=(H/L)^{-0.5}\tan\beta$ (The Iribarren Number) and Δ is the relative density of the revetment.

Linked block revetments

For linked blocks, Pilarczyk (1990) should be used:

$$D = \frac{\phi}{\Delta}\frac{0.035}{\psi_{cr}} K_T K_h K_s^{-1} \frac{U_d^2}{2g} \qquad \text{(D.13)}$$

where D is the characteristic size of protection, ϕ is a stability correction factor, Δ is the relative density of the revetment, ψ_{cr} is stability factor, K_T is the turbulence factor, K_h is the depth factor (D/y)^0.2 where y is the water depth (at the toe of the bank), K_s is the slope factor (which is defined by soil internal friction, bank slope and channel slope) and U_d is the depth-averaged flow velocity.

Although the linking of the blocks should provide extra support against block uplift it should not be assumed in design (Escaremia, 1998). However the linking of blocks does provide extra resistance against sliding which is given by:

$$F_{fl} = [l_3 \times b \times t_a \times \rho_c \times g \times \cos\alpha] \qquad \text{(D.14)}$$

where l_3 is the length of cable-tied revetment above the water line.

Bitumen bound revetments

Bitumen can be used to bind rocks and stones together to make the revetment more stability, but also presere some flexibility, they are sub-divided according to their permeability. The minimum layer thickness should be about 2 to 3 times the maximum stone size.

Permeable revetments

A simple formula for calculating the cover layer thickness, t, can be used:

$$T_a = C \times H_s \tag{D.15}$$

where C is a coefficient dependent on the sub-base. Permeable revetments should also be tested for wave impact pressures if they are expected to be subjected to these.

Impermeable revetments

The two main failure mechanisms for impermeable bitumen-bound revetments are uplift and sliding. The uplift pressure is calculated using Van der Meer's method (RWS, 1985):

$$p_u = \upsilon \times \sqrt{\left[1 - \frac{\upsilon}{a+\upsilon}\right]^x} \tag{D.16}$$

where x is dependent on the revetment slope , v is the vertical distance between the water level and phreatic surface, a is the vertical distance from the toe to water level.

The maximum uplift pressure is therefore:

$$\sigma_{w0} = \rho_w \times g \times (p_u + t_a \times \cos\alpha) \tag{D.17}$$

to check the revetment for stability against uplift:

$$t_a \geq \sigma_{w0} / \rho_a \times g \times \cos\alpha \tag{D.18}$$

to check the stability against sliding:

$$t_a \geq f \times \sigma_{w0} / \rho_a \times g \times (f \times \cos\alpha - \sin\alpha) \tag{D.19}$$

where f is the coefficient of friction which equals tanϕ' if ϕ'>θ, or tan θ in other conditions, where θ is the angle of friction between revetment and soil and ϕ'=angle of internal friction of sub-soil.

Wave impact loading

A method for design against wave impact is given by RWS (1985). The maximum wave impact force, P_{imax}, is given by:

$$P_{i\max} = b_i p_{\max} = b_i q_i \rho_w g H_s \tag{D.20}$$

where p_{max} is the maximum pressure, b_i is the width over which the pressure acts (=0.4H_s) and q_i is a factor dependent on revetment slope. To calculate cover layer thickness:

$$t_a \geq 0.75\ ((27/16) \times (1/(1-\upsilon^2)) \times (P_i / \sigma_b)^4\ (S/c))^{0.2} \tag{D.21}$$

where σ_b is the asphalt failure stress, S is the asphalt stiffness modulus, υ is the asphalt's poisson ratio (=0.35), c is the modulus of subgrade reaction. However it should be noted that this is only the design thickness for one loading. In order to compensate for the repeated loading of differing waves a fatigue factor should be applied, although a lot of wave data is required to do this very accurately, a simpler method is to use:

$$f_f = \left[0.1 \times N\left\{\sum {n_i}/{n_s}\left({P_i}/{P_s}\right)^5\right\} + 1\right]^{4/25} \quad \text{(D.22)}$$

when the extreme conditions are the design conditions, if the revetment is not exposed to design conditions under an extreme event,

$$f_f = \left[0.1 \times N\left\{\sum {n_i}/{n_s}\left({P_i}/{P_s}\right)^5\right\}\right]^{4/25} \quad \text{(D.23)}$$

should be used.

Other revetment types

Geomats

These are synthetic mats that are able to retain soils and also allow vegetation to grow through them. The mats are normally only a few millimetres or centimetres thick and usually anchored at their top or bottom.

Concrete mats

The suggested thickness of concrete mats is the same as that for block revetments.

Flexible forms

These are geobags or geotubes filled with sand or gravel (or cement, perhaps) that can be used as revetments. They are usually employed as emergency temporary measures, and as there are no generic guidelines the manufacturer should provide the information about their product.

Piling

This should be analysed as shown in the section on vertical retaining walls in Section D.8.

D.5.5. Filter design

Filters should increase in permeability from the subsoil to the cover layer thereby ensuring that there is not an excessive build-up of hydraulic pressure, whilst preventing washing out of material from layers beneath.

Granular filters

The filter must obey some filter laws:

- $D_{15\,\text{filter}} < 5\, D_{85\,\text{base}}$ Stability or piping criterion
- $D_{15\,\text{filter}} > 5\, D_{15\,\text{base}}$ Permeability criterion
- $D_{50\,\text{filter}} < 25\, D_{50\,\text{base}}$ Uniformity criterion

Geotextile filters

An additional concern with geotextile filters is deterioration by uv light or reaction with chemicals in the water passing through it. To meet the permeability criteria, the geotextile should satisfy:

$$k_g \geq 5k_s \qquad \text{(D.24)}$$

where k_g is the permeability of the geotextile and k_s is the permeability of the underlying material.

D.5.6. Additional details

Overtopping

This is related to the structures freeboard above design water level and also the properties of the design wave.

Scour

This is related to the design wave, the structures slope and type of revetment chosen. Toe protection should extend a distance of at least the maximum wave height below the toe.

D.6. NATURAL COASTAL PROTECTION

Natural coastal defence refers to non-structural means of reducing erosion or flood risk. This can take the form of an entirely beach based defence (such as shingle banks) or in addition to structural solutions (such as nourishment of the beach in front of a seawall resulting in decreased loading on the structure). The level of protection therefore offered by natural forms of protection will be a function of time and the antecedent loading conditions. Natural protection does not always fail in the absolute sense of the word, more likely it will offer a reducing (although given suitable conditions this can increase) level of protection (or performance) over a given time period. D'Angremond (1992) suggested that the definition of failure for a natural protection scheme is when the scheme fails to provide the require level of protection after a significantly short period of time than that designed for.

D.6.1. Dune management

A soft form of coastal defence, dune management can be a cost-effective and environmentally sound means of defending sandy coastline. The dunes serve two purposes, first as an embankment against flooding and secondly as a reservoir of sand, replenished when beach levels are high and able to provide a feed of extra material during storms (CIRIA, 1996). Dunes normally form when sand is trapped on the windward side of a plant or piece of litter, this can then build up. More vegetative matter provides additional stability and more chance of additional sand being caught on the dune.

Dunes are eroded by wind or waves. Wave induced erosion occurs when the sea is at the base of the dune and causes failure by toe scour. Wind induced erosion is rarely caused by wind alone, but more likely it exaggerates other damage (usually to the vegetation) caused by humans and other animals.

Good dune management should be included in a beach management programme as any attempt to maintain just the dunes in a fast eroding beach will not be successful. The use of sand fences and dune armour can help encourage dune growth and protection. One of the most complex factors in dune management is its effect on adjacent frontages; as it is impossible to simply terminate a dune system their can be quite a few complex interactions. The most threatening of which is the possibility of erosion at the junction, this could lead to flooding behind both types of defences. The following six points should be considered when designing a dune management system:

(1) Effect of adjacent defences on sand supply to beach/dune system (*eg.* caused by groynes),
(2) Consequences of sudden, temporary retreat of dune/beach system on adjacent coast,
(3) Effects of wind-blown sand on adjacent frontage,
(4) Danger of access and disturbance to dunes and their vegetation cover,
(5) Wind induced onshore transport needs to be at least equal to wave induced off- and longshore transport, and,
(6) Volume of material that would be removed in a storm (influenced by wave properties, duration of storms, water levels, slopes, sand grading).

D.6.2. Beach nourishment

A healthy beach is one of the most effective forms of coastal defence as it has the ability to evolve naturally depending on the wave and tidal conditions. Beach nourishment is the addition of sand to the beach in order to replenish losses, increase the level of defence protection or provide recreational value to the area. Sometimes it is necessary to reduce the sediment losses from the beach by either using beach control structures or recycling sand from the downdrift end of the beach.

The design of beach nourishment material is done based on the grain size at the site in question. It is usually the case that a coarser grain than that already there will be used (although the designer must be careful that smaller particles are not washed out). The other design factor is the beach slope and there are a range of stable slopes depending on the grain size employed.

Choosing the design volumes to be used for nourishment is usually based on a combination of two approaches:

- Empirical approaches based on past experience (particularly where nourishment has been ongoing for some time)
- Modelling methods (which are a combination of empirical and analytical information because data is required for calibration) based on predictions of longshore and cross-shore beach response (particularly where a new or changed beach nourishment programme is planned)

The design of the profile of nourishment for sand beaches has many suggested methods and it has been suggested that no one method should be used in isolation (Davison, Nichols and Leatherman, 1992). Dean's method (Dean, 1991) determines the volume of nourishment required to increase the beach width by a certain amount. The Pilarczyk, van Overeem and Bakker (Pilarczyk *et al.*, 1986) method considers the reshaping of the beach profile in response to prevailing hydraulic conditions and the depth to which this profile develops. The 'overfill ratio' method quantifies the amount of excess nourishment that is required to mitigate losses from not having the ideal mix of grain sizes for a given location (James, 1975)

The design of shingle beaches should not use the same technique and an equilibrium technique was derived (Powell, 1993) for shingle beaches of a dissimilar grading. This method was developed at HR Wallingford and at present not enough comparison with real field data has been possible (CIRIA, 1996).

The key parameters in beach nourishment design are the grain size, D, the significant wave height, H_s. Depending on the design methodology used, other wave properties (such as period) and beach dimensions may be required.

D.6.3. Managed retreat or landward realignment

Often instead of advancing or maintaining the present beach, it is more beneficial to manage a landward realignment. This is often used to obtain a more favourable cost-benefit option whilst minimising interference with natural processes. Often this is preferential to allowing coastal squeeze which could result in the loss of the intertidal habitat.

Landward realignment can take one of four forms; the setting back of the line of defences further inland, controlled abandonment allows the sea to form its own new water edge (Note: this is not equivalent to the 'do nothing' option as this requires monitoring to ensure no unwanted erosion). The other forms of retreat are to deliberately reduced the size of the defence or create a tiered form of defence when the level of protection is gradually increased further inland.

Landward realignment can also be used to environmental gain as saltmarsh or mudflats can be created on what might have previously been low-lying farmland. Retreat can also be used to help provide a sediment source for further downdrift in order to provide more protection.

D.6.4. Rock beaches

The principle role of a rock beach is to dissipate wave energy. In doing so the structure is expected to alter shape, but must not allow excessive overtopping of the beach crest, significant seepage through the beach ridge or too great a loss of material. The main design problems are (CIRIA, 1996):

(1) Cross-shore movement, although this is can be beneficial as the crest can build upwards increasing overtopping resistance under all but the most severe storms.
(2) Longshore movement, which can lead to degradation and loss of material from the updrift parts of the structure.
(3) Material loss resulting from abrasion/attrition causing either a loss of material or a wider grading of material reducing the beaches resistance to wave action.

Principal parameters that influence hydraulic performance and beach stability are (CIRIA, 1996 and CIRIA and CUR, 1991):

(1) Rock parameters
    a. Grading D_{85}/D_{15}
    b. Strength of rock and resistance to attrition
    c. Median rock size: D_{n50} $(M_{50}/\rho_r)^{1/3}$ (*M* is the mass of individual rocks not exceeded by 50% of all rocks in a given grading)
(2) Hydraulic parameters
    a. Relative wave height: $N_s = H_s/\Delta Dn_{50}$
    b. Relative mean wave period: $T_o = T_m/(g/\Delta Dn_{50})^{1/2}$
    c. Wave angle to the shoreline: β
(3) Beach dimensions
    a. Thickness of permeable beach: t_a
    b. Initial slope: α
    c. Crest freeboard relative to design water level: R_c
    d. Permeability of beach to wave action (given by internal penetration length: λ_l which is the penetration of wave agitation across the internal water table, it is dependent on porosity, permeability, loading rate and wave period)

Rock slope movement is predicted using BREAKWAT (Van Der Meer and Pilarczyk, 1986) or SHINGLE (Powell, 1990) software which both relate profile geometry to wave parameters. This can allow the designer to quantify the amount of rock required for a specific design life.

Mudflats and Saltmarshes

Aside from their environmental value mudflats and saltmarshes are a highly effective means of dissipating wave energy and 10m of saltmarsh allows the height of the flood defence to be halved (National Rivers Authority and Binnie and Partners, 1993). The main causes of erosion are a rising sea level, reduced sediment supply, natural cyclical pattern, pollution and increased wave activity. Cohesive elements tend to be lost in blocks, where as granular materials are eroded more slowly.

D.6.5. Other methods

Sills

These are un-segmented, always or occasionally submerged shore parallel structures that are designed to reduce inshore wave climates by inducing waves to break. They have so far only been seen to be useful in low to moderate wave energy, micro tidal environments with low tidal drift (CIRIA, 1996).

Beach drainage

This involves the artificial lowering of the water table in the intertidal zone. This enhances the wave energy absorbing capacity of the beach. This can be achieved by pumps or drains, but has yet to be proven in a large tidal range and in storm dominated wave conditions.

Supply restoration

This involves the restoration of sediment supply or littoral drift so as to maintain balanced sediment cell budgets within a management unit. This can be required when a man-made structure has dramatically altered the sediment flow regime causing a sediment deficit in cells along the coastline. Supply can be restored by alteration to existing structures or standard nourishment at the appropriate place.

Sediment bypassing

This involves the movement of material of material from an accreting length of shoreline to an eroding length of shoreline. This can be done by mechanical bypassing when lorries transport the sand from one area to another. Hydraulic bypassing involves pumping the sand as a slurry. Seabed fluidisation increases the amount of material for bypassing; by pumping water at high pressures in the seabed, sand is released to be pumped away. There are other bypassing systems that are not well tested but the methodology used should be site specific.

D.7. RIVER BANKS AND LEVÉES

D.7.1. Brief Summary

Not all river banks need artificial protection and often good river management is the optimum solution. Levees are river banks that have been raised to contain high water levels.

D.7.2. Processes effecting failure

The failure of a river bank can be considered in terms of overall stability and also local stability. It should be noted that local in-stability can lead to mass bank failure. Failure modes of non-revetted slopes are predominantly governed by soil parameters such as shear strength and pore water pressures. The factors that determine stability in mass bank failure are:

(1) Surface water and groundwater regime
    (a) Seepage

Excess pore pressure in the bank can trigger a rotational failure, as well as encourage erosion, toe scour and piping.

(b) Infiltration

Water infiltrating the soil can cause the unit weight to increase, combined with an increase in pore pressure the bank is more prone to failure.

(2) Surcharge loading

Loading on top of a bank increases its susceptibility to mass failure.

(3) Tension cracks

Formation of these in cohesive soils reduces stability particularly if filled with water.

(4) Vegetation

Soil properties can be improved in particular the shear strength, as well as possible provision of some tensile strength thereby increasing stability of the bank.

(5) Toe scour and surface erosion

Toe scour increases bank height and causes undercutting whilst a change in slope geometry caused by either scour or surface erosion can alter bank stability.

D.7.3. Failure modes

Planar failure

This is failure of a bank along a plane surface, this sort of failure is normally associated with non-cohesive soils or materials in which relatively deep tension cracks have developed. Safety against planar failure is analysed as follows:

(1) Consider the stability of a single wedge or slab as shown Figure D.6.

(2) Assume a potential failure plane and calculate FoS.

(3) Forces to consider:

(a) Weight of slab and hence normal stresses on failure plane.

a. Shear forces acting the plane to resist failure.

$$FoS = \frac{2Cu \sin\alpha}{\gamma s H \sin(\alpha - \beta)\sin\beta} + \frac{\tan\phi}{\tan\beta} \qquad \text{(D.25)}$$

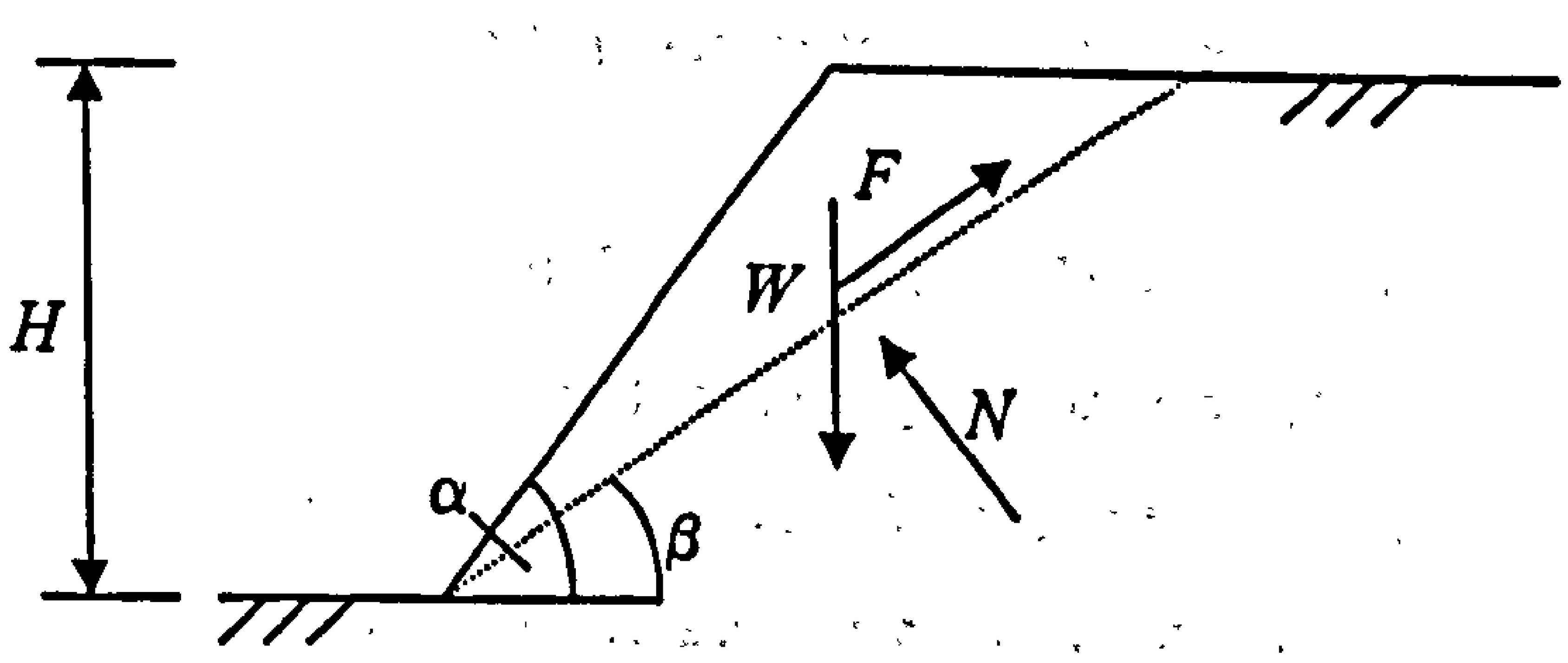


Figure D.6 Planar slope failure of an embankment

Modifications to the theory can be made to allow for tension cracking allowing the critical height of the bank to be assessed. A number of failure planes should be analysed to ensure the lowest FoS is obtained.

Shallow failure

This type of failure occurs parallel to a shallow bank which consists of predominantly non-cohesive materials. Safety against shallow failure is analysed as follows:

(1) Consider the bank as an infinite slope as in Figure D.7. Where the failure surface is at a depth that is small compared with the length of the slope.

(2) A slice can be taken and as all slices will be identical any interslice force will be equal and opposite and can therefore be ignored.

(3) The water table is assumed to be parallel to the slope and steady seepage is taking place in a direction parallel to the slope.

(4) $\tau_f = c' + (\sigma - u)\tan(\phi')$ and $FoS = \tau_f/\tau$

(5) where $\sigma = \{(1-m)\gamma + m\gamma_{sat}\}z\cos^2\beta$; $\tau = \{(1-m)\gamma + m\gamma_{sat}\}z\sin\beta\cos\beta$; u(on plane) $= m\,z\,\gamma_w \cos^2\beta$

(6) For a total stress analysis the values of C_u and ϕ_u are used with u=0.

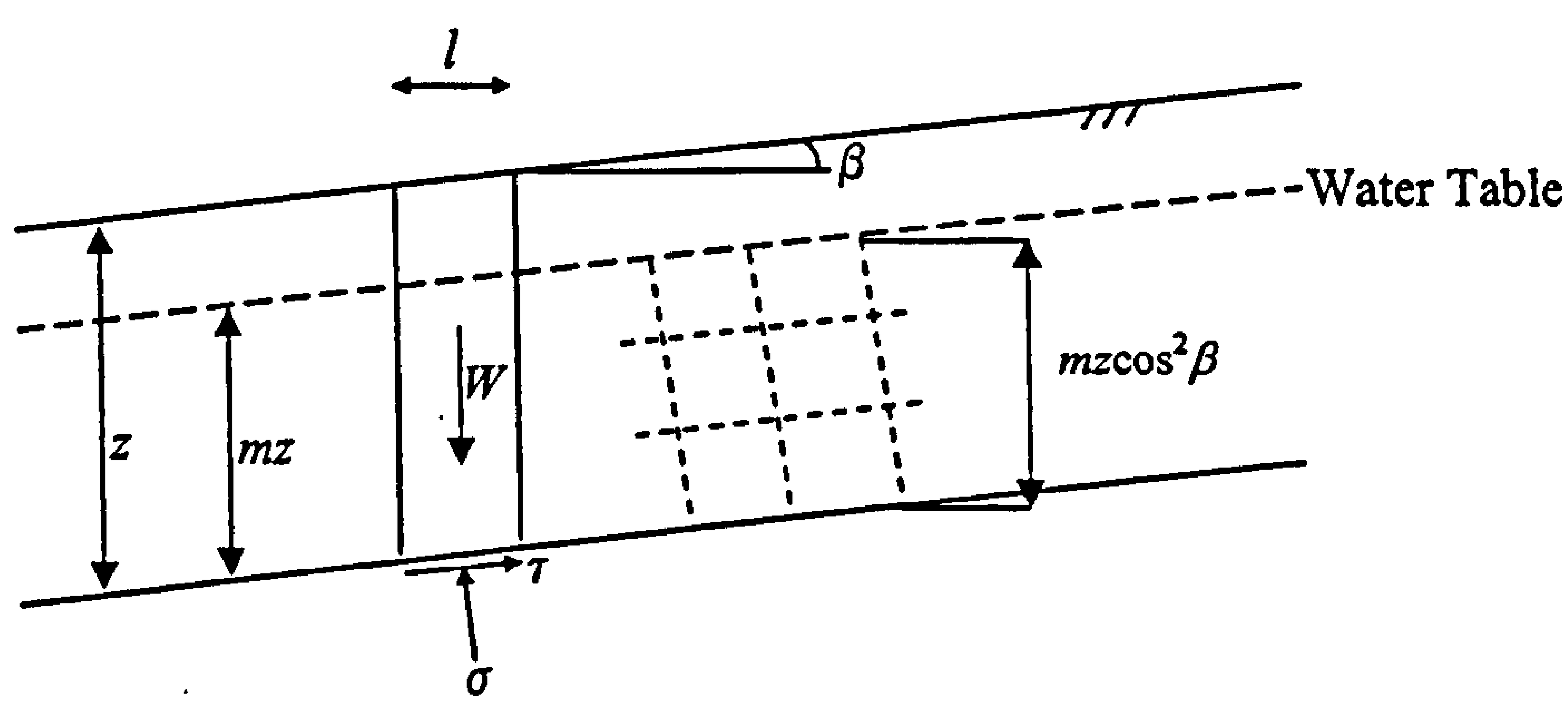


Figure D.7 Shallow slope failure of a slope

Rotational failure

This type of failure is most likely on a steep (and high) bank consisting of cohesive material (if the bank is homogenous then the failure plane will be rotational as in Figure D.8, unhomogenous banks may have spiral or planar elements of their surface). Safety against rotational failure is designed for as follows.

(1) As with planar analysis a number of potential failure surfaces should be analysed.

(2) The easiest way to perform the analysis is to assume a circular plane of failure (as shown in Figure D.8)and that the soil is purely cohesive and homogeneous.

(3) By dividing the surface into sections and assuming the interslice forces cancel out the shear force along the bottom of the slice and the weight can be equated.

$$FoS = \frac{c'La + \tan\phi'\Sigma N'}{\Sigma W\sin\alpha} \qquad \text{(D.26)}$$

where L_a is the arc length and N is the normal force on the base (σl).

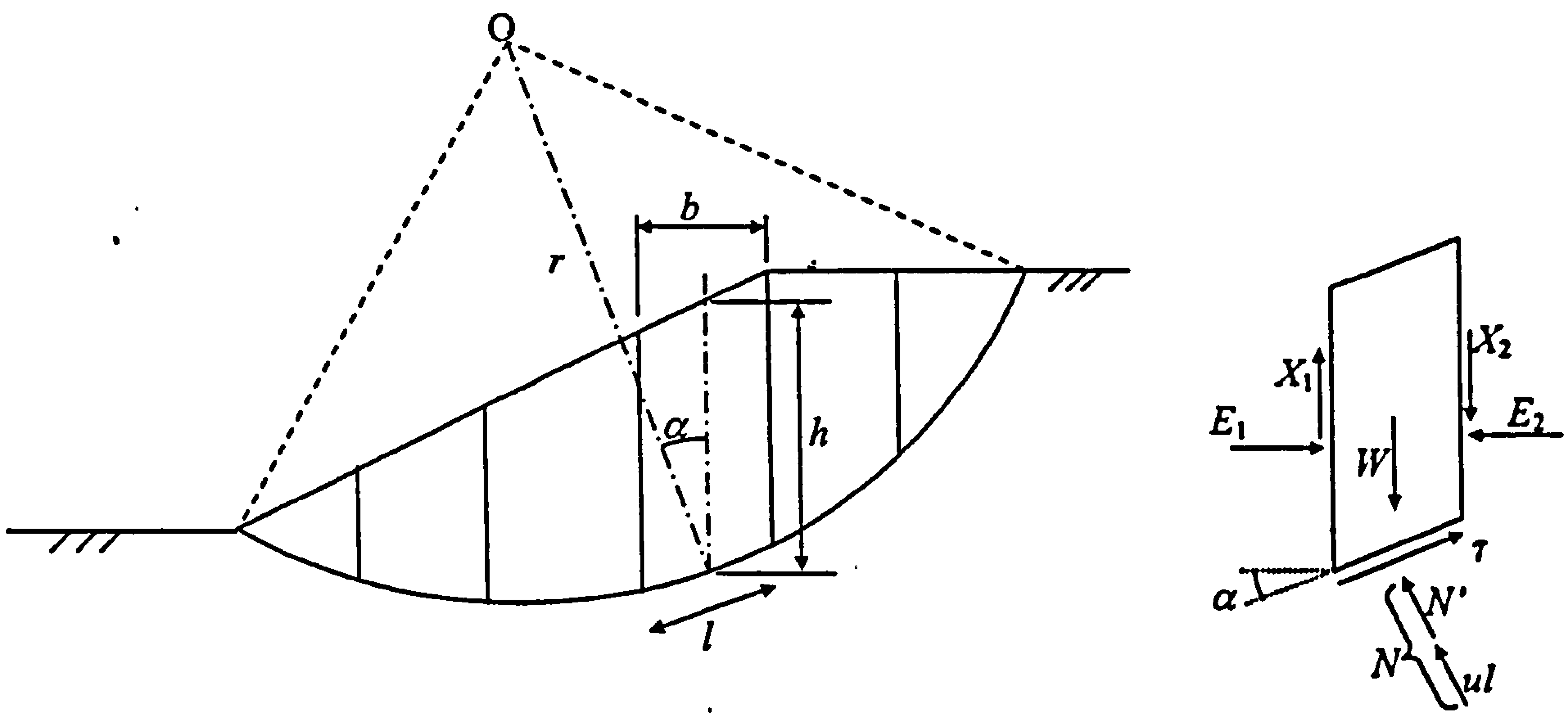


Figure D.8 Rotational slope failure and division into slices used for failure analysis

(4) Equation D.26 used to estimate the FoS is exact, approximations are introduced when estimating N'.

(5) Two methods can be used to solve this equation, the Fellenius solution assumes the resultant of the interslice forces is zero and therefore $N'=W\cos\alpha - ul$. This method underestimates the FoS within 5-20%.

(6) Bishop's method assumes that the resultant forces on the sides of the slices are horizontal meaning that:

$$\tau = \frac{1}{F}(c'l + N'\tan\phi') \tag{D.27}$$

(7) Resolve in the vertical direction:

$$N' = \left(W - \frac{c'l}{F}\sin\alpha - ul\cos\alpha\right) \Big/ \left(\cos\alpha + \frac{\tan\phi'\sin\alpha}{F}\right) \tag{D.28}$$

(8) Substituting this into Equation D.26 it can be seen that the FoS appears on both sides and so the solution has to be converged upon therefore making this method more suitable for computational analysis.

Composite failure

Failure of composite banks will depend on the nature and thickness of the layers as well as their relative positions. If soil conditions are correct then the bank can fail as described previously and so the appropriate methods can be use.

In low composite banks an undercutting can be formed creating an overhang which can then fail in shear, tension or as a beam. Shear failure is uncommon, but the final choice of failure mechanism is not based on soil properties but the cantilever geometry and soil irregularities. The three types of composite failure are:

a) Tension failure

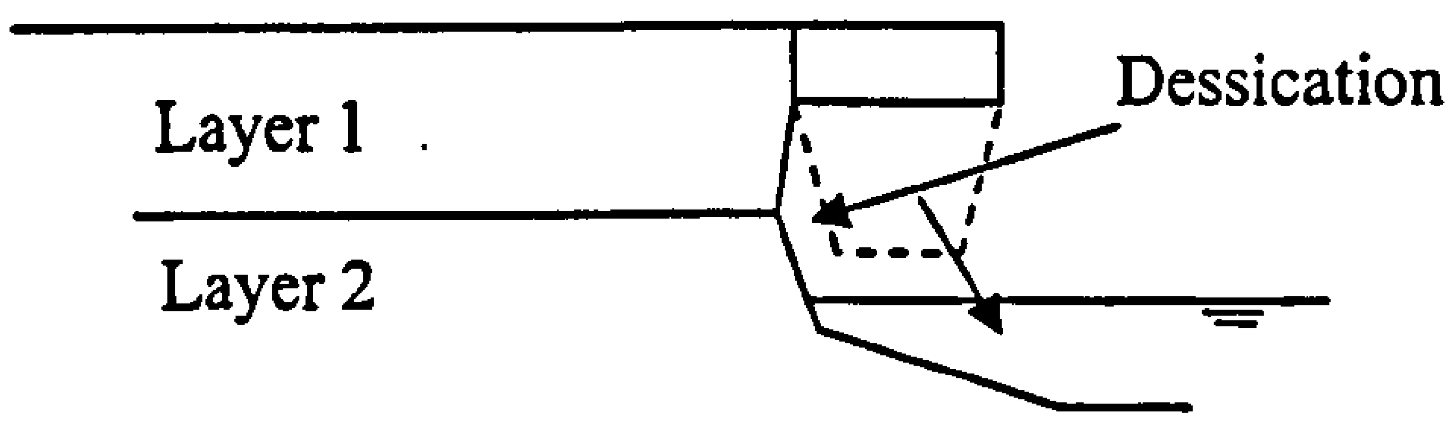


Figure D.9 Tension failure of river embankment

b) Shear failure

This form of failure is rare with little available information (Hemphill and Bramley, 1989)

c) Beam failure

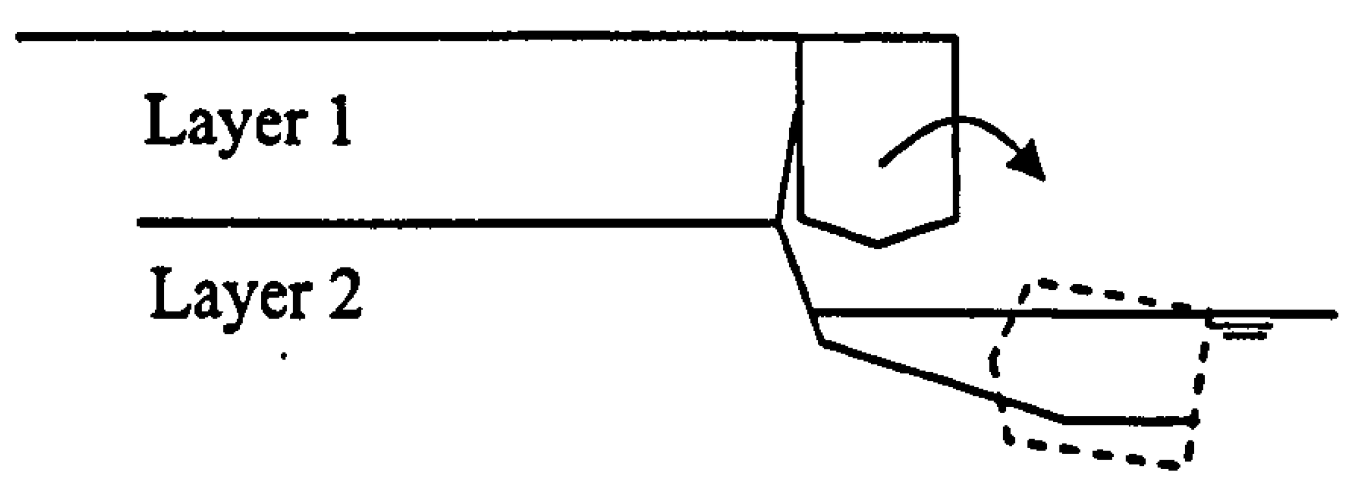


Figure D.10 Beam failure of river embankment

No source of information which performs a quantitative analysis of tension failure of river banks has been found.

Flotation

This is a rare failure mode, but is seen when unstable soil (such as peat) has been used in the bank construction

Erosion

This is gradual failure process (and often leads to other types of mass failure). It is influenced by many different loading processes which are described below.

<u>Current flow</u>

The boundary shear stress in a channel of finite width is given by: $\tau = \gamma RS$ (where R is the hydraulic radius, S is the bed slope and γ is specific weight of water). A higher shear stress will give rise to more scour on the river bed. The Shield's formula (Equation D.4) gives an estimation of the critical shear stress for a particle on a river bed.

<u>Irregularity in channel</u>

This can cause the creation of higher shear stresses and secondary currents which increase scour. Any local feature (such as a culvert, bridge pier or vegetation) can alter the local flow pattern potentially causing downstream erosion. The variation in shear stress can be measured in the field or by using the assumption that the velocity profile is logarithmic and the Karman-Prandtl equation applies:

$$v = 5.75\left(\frac{\tau}{\rho}\right)^{1/2} \log y + const. \quad \text{(D.29)}$$

where v is the velocity at distance y from the bed and ρ is the density of water. Another relationship used is that comparing U to U_b:

$$U_b = \frac{U}{0.68\log(y/k_s)+0.71} \quad \text{(D.30)}$$

To predict maximum scour depth in meandering channels, is done using a formula proposed by Apmann (1972):

$$\frac{d_{max}}{d_0} = \frac{(n'+1)(B/r)}{1-(1-(B/r))^{n'+1}} \quad \text{(D.31)}$$

Where d_0 is the mean depth, r the outer bank radius, B the channel width and n' is a coefficient (often equal to 2.5, but this is assigned from limited data)

Wave action

Wave length is calculated differently depending on the depth of water, for deep water:

$$L_0 = \frac{gT^2}{2\pi} \quad \text{(D.32)}$$

for water of an intermediate depth:

$$L = \frac{gT^2}{2\pi}\tanh(2\pi d/L) \quad \text{(D.33)}$$

and, for shallow water:

$$L_s = T\sqrt{gd} \quad \text{(D.34)}$$

where L_o, L and L_s are the wave lengths in deep, intermediate and shallow water respectively, T is the wave period and d is the depth of the water. If $d<1.28H$ approximately then the wave will break before hitting the bank, this is generally less damaging than if they break on the bank. The damage potential of a wave depends on whether the wave plunges, collapses or spills against the bank. This behaviour depends on the Iribarren number (I_r):

$$I_r = \frac{\tan\alpha}{(H_s/L_0)^{1/2}} \quad \text{(D.35)}$$

where α is the banks slope.

Wind generated

For a river wind waves are normally fetch-limited and so the length of the wind gust is not considered. A recommended formula for estimating significant wave height is the Sverdrup-Munk-Bretschneider equation:

$$H_s = 0.00354(U_{10} / g)^{0.58} F^{0.42} \qquad \text{(D.36)}$$

Boat generated

The waves generated depend on the size and geometry of the boat. If the boat speed, V_b, is:

$$V_b \leq 0.7\sqrt{gd} \qquad \text{(D.37)}$$

then the corresponding wave length is:

$$L_b = \frac{4\pi}{3}(V_b^2 / g) \qquad \text{(D.38)}$$

If this speed is exceeded, then the wave heights are primarily determined by the boat's speed, shape and size and the depth of the water. Boats with propellers can increase local flows, these effects are most pronounced when the boat is performing a complicated manoeuvre or starting. The scour depends on many factors including the type of engine, propeller sizes and duration of power burst.

Mechanical erosion

Freeze-thaw cycle. Internal friction is reduced by water freezing and reducing particle packing, the consequential thawing results in a weakened soil.

Desiccation. Clay soils swell and shrink during wetting and drying cycles causing vertical fissures thereby weakening the soil.

Boat impact. Boats mooring and crashing, but also insertion of mooring pegs can weaken the soil or directly cause damage.

Animal and man. Surface trampling reduces vegetation and therefore surface protection meaning the bank is more prone to erosion, burrowing can reduce the bulk strength of the soil.

Seepage

Seepage velocity depends on the local hydraulic gradient, seepage flow into the channel can increase the lift force on the surface material thereby increasing its susceptibility to removal. This can effect a semi-permeable soil considerably if there is a sudden draw down, or a natural (*eg.* tidal) cycle.

Piping occurs when the net effective stresses are zero and is more likely to occur when beds of highly permeable soil are surrounded by beds of less permeable soil.

Suffusion (removal of fine material from a network of coarse particles) can occur when the fabric is incompletely filled by fines.

High pore water pressure gradients are less likely to occur in highly permeable banks meaning that they are less likely to suffer from seepage erosion problems. On the other extreme highly impermeable clay banks will have extremely low seepage velocities and so not suffer from seepage problems.

Surface run-off

If the infiltration capacity of the bank is exceeded then surface runoff occurs, causing increase in damage to the soil surface.

Particle entrainment

The *composition* and *cohesiveness* of a river bank influence its rate of erosion. Cohesiveness depends on the degree of compaction and the organic content and is therefore highly variable. For non-cohesive beds, the critical shear stress for particle motion is calculated using the Shields (1936) formula (Equation D.4).

Lane (1955) developed a formula for critical shear stress for a river bank, τ_0:

$$\frac{\tau_0}{\tau_c} = \left(1 - \frac{\sin^2 \alpha}{\sin^2 \phi}\right)^{1/2} \qquad \text{(D.39)}$$

where τ_c is the critical shear stress of the bed, α is the bank slope and ϕ is the angle of repose of the soil material. Cohesive banks require higher stresses to have soil particles removed due to the effects of physical cohesion and electro-chemical bonding.

The presence of *vegetation* can reduce local stress by affecting velocity gradients on the bank and increase erosion resistance of the surface material. However 'rigid' vegetation (such as trees) can affect the local flow pattern in the channel particularly when its roots are exposed or when it falls into the river. This can lead to an increase in local scour and a disruption to the flow pattern downstream.

D.8. VERTICAL RETAINING STRUCTURES

D.8.1. Brief summary

These structures are designed to provide vertical reinforcement for a river bank. Banks that are steeper than the soils angle of repose may be prone to failure. Vertical protection is designed to resist erosive forces as well as soil and groundwater pressure from the channel bank. The main types of retaining structure are:

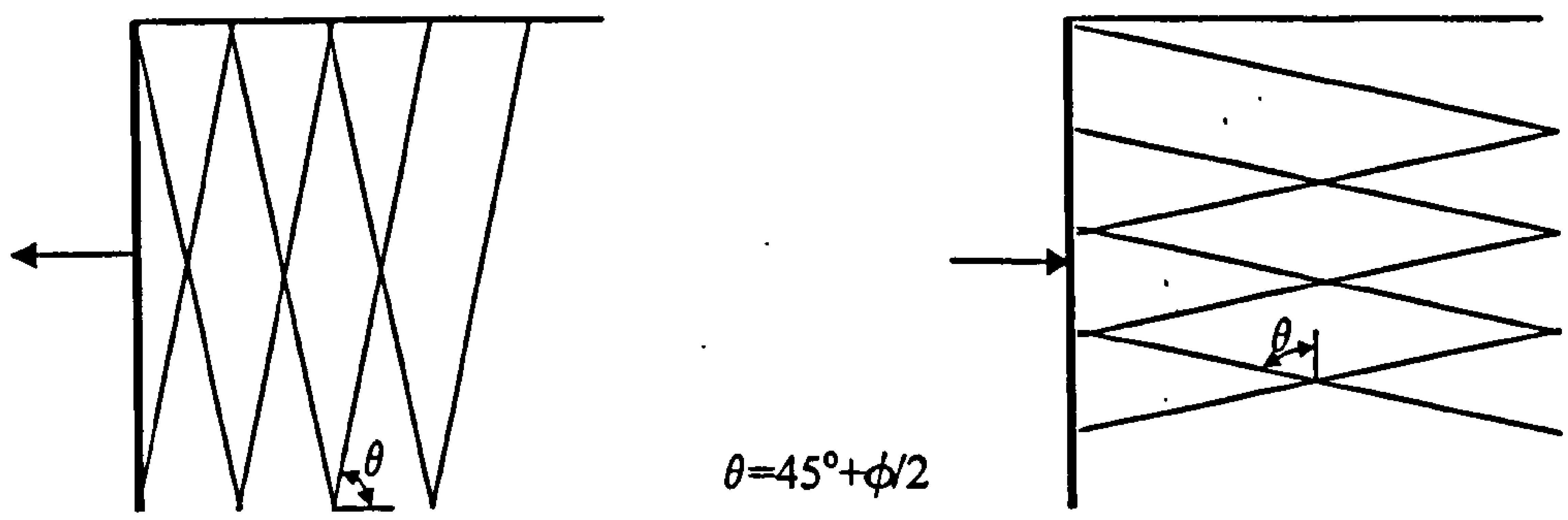
(1) Gravity walls

(2) Gabion walls

(3) Cantilever walls

(4) Sheet piles

Specific design issues for these structures are considered later in this section. A few key points of geotechnical theory used in the design of vertical retaining structures are now described. Figure D.18 shows a semi-infinite homogeneous and isotropic mass of soil with a vertical boundary formed by a smooth wall surface.



Active Rankine state   Passive Rankine state

Figure D.11 Active and passive Ranking states

A soil element at depth z has stress σ_z acting on it in the vertical direction and σ_x in the horizontal direction. The active, p_a, and passive pressures, p_p, are given by:

$$p_a=K_a\gamma z - \sqrt{K_a}\,,\ p_p=K_p\gamma z - \sqrt{K_p} \tag{D.40}$$

When the horizontal stress becomes equal to the active pressure the soil is in an active Rankine state, conversely when the horizontal stresses become equal to the passive pressure the soil is in a passive Rankine state. The active, P_a, and passive force, P_p, per unit length of wall is given by integrating the pressure distributions, p_a and p_p, over the depth, H, of the wall:

$$Pa = \int_0^H p_a dz, Pp = \int_0^H p_p dz \tag{D.41}$$

K_a and K_p are the active and passive earth pressure coefficients respectively which are defined by:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi}, K_p = \frac{1+\sin\phi}{1-\sin\phi} \tag{D.42}$$

D.8.2. Gravity walls

These are large solid structures that support the bank by use of their self-weight and frictional forces generated along their base. They can fail in many ways:

Bed Scour

Scour in front of the wall causes loss of bed restraint and foundation support leading to sliding, bearing or overturning. Scour of foundation can be prevented by use of a cut-off

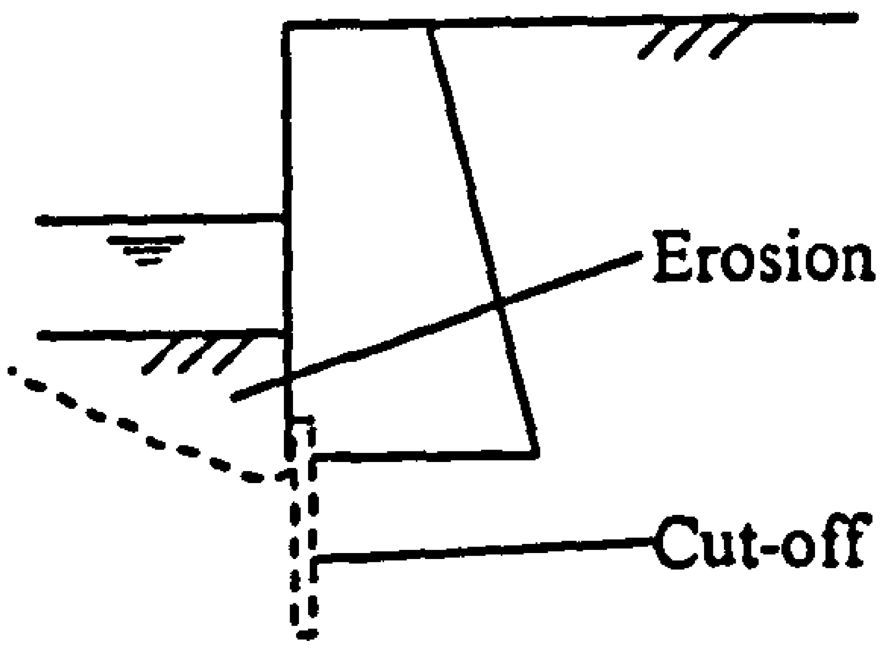


Figure D.12 Gravity wall failure from scour

Sliding

Occurs when combined surcharge, active soil pressures and hydrostatic forces exceed restraining forces. Sliding resistance at base can be increased by shear key.

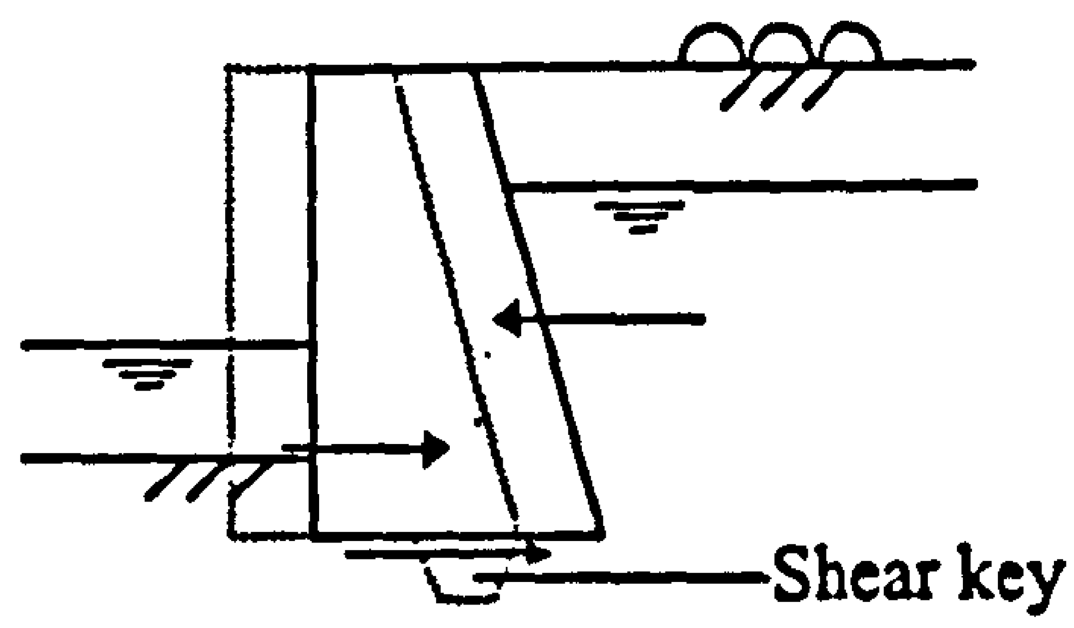


Figure D.13 Gravity wall failure from sliding

Overturning

Occurs when sum of overturning moments exceeds sum of restoring moments. Can be aided by drop in water level or loss of toe material.

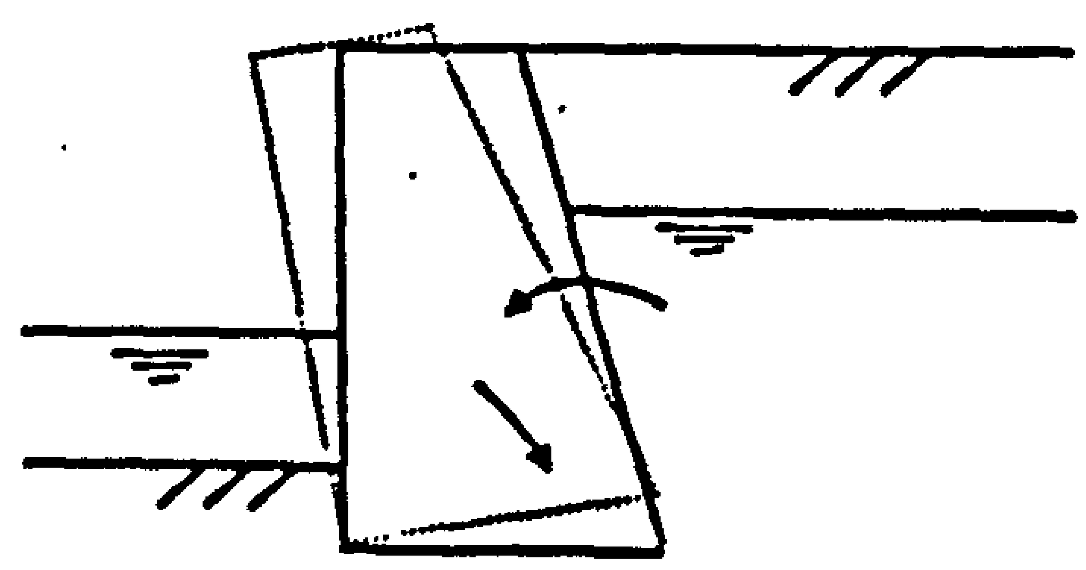


Figure D.14 Gravity wall failure by overturning

Piping

Occurs when a hydraulic gradient leads to seepage causing piping under foundation that potentially leads to bearing failure.

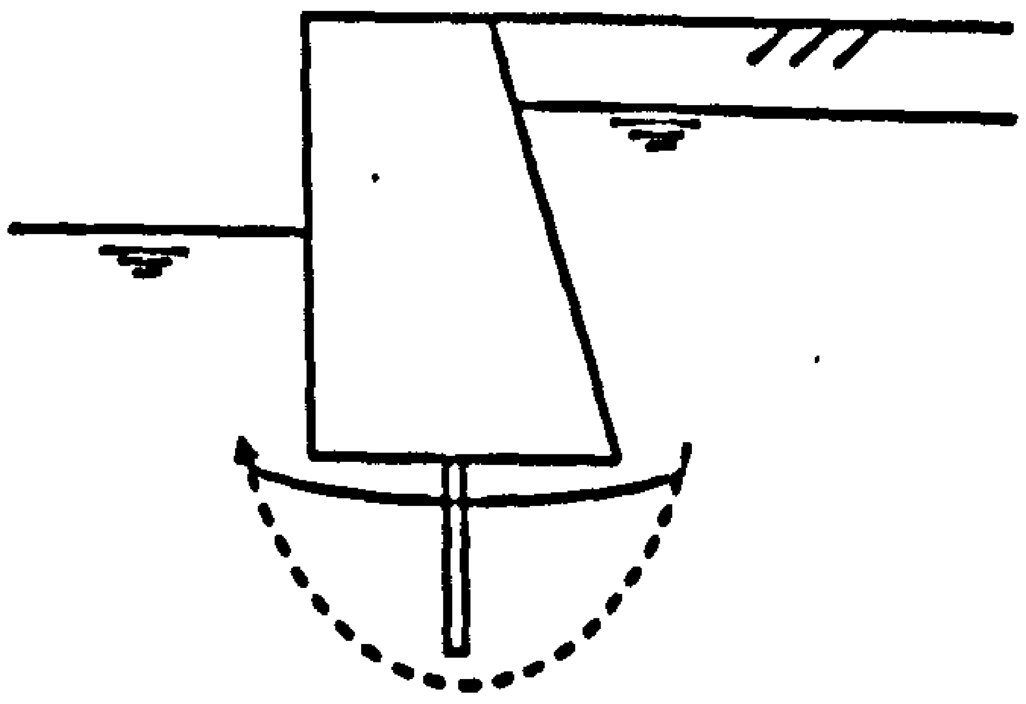


Figure D.15 Gravity wall failure from piping

Bearing pressure

Occurs when bearing pressure at toes exceeds limiting pressure, resulting in collapse or unacceptable movement.

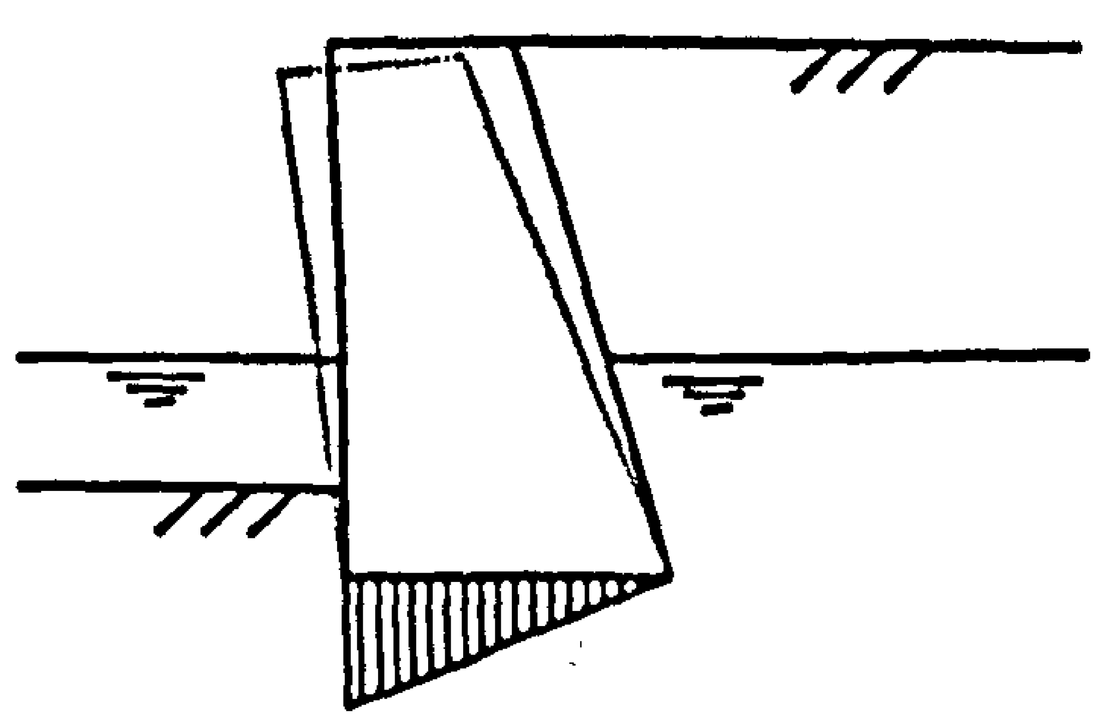


Figure D.16 Gravity wall failure due to bearing pressure

Rotational slip

Low shear strength (possibly caused by lowering of water level) or high surcharge can lead to rotational slip.

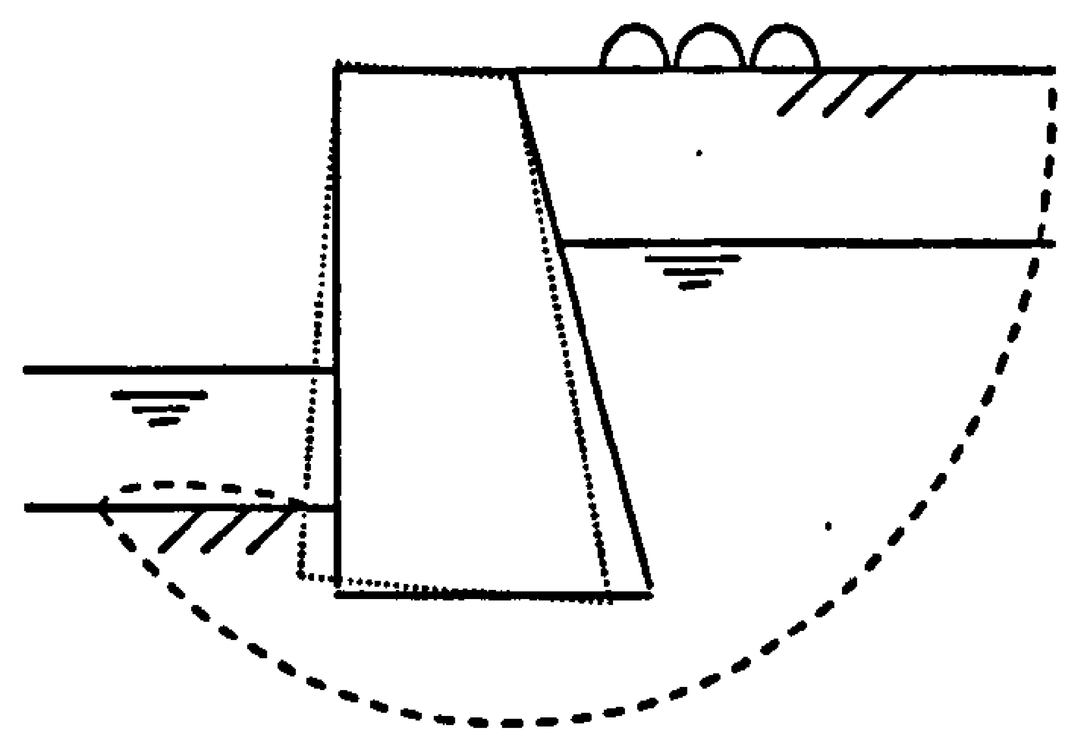


Figure D.17 Gravity wall failure by rotational slip

The final failure mode of a gravity wall is due to tension cracking in the rear face of the wall following leaching of mortar and/or decomposition of material. Allowance for a degree of vandalism and abrasion should also be considered in design.

Design

These walls derive stability from self weight. Geotechnical factors are the most important in terms of failure analysis and are considered by resolving forces and moments. These should be analysed at normal and the most extreme conditions of seepage and water height variation. The structure should be shaped so that the resultant downward force acts within the middle third of the base length (so as to avoid any of the base being in tension). As with all vertical retaining structures, it is important to be very careful when making bed level assumptions, both the long and short term level can have a major impact on stability.

D.8.3. Cantilever walls

These are designed with the same considerations as a gravity wall, but the soil above the toe (Figure D.18) provides additional stability.

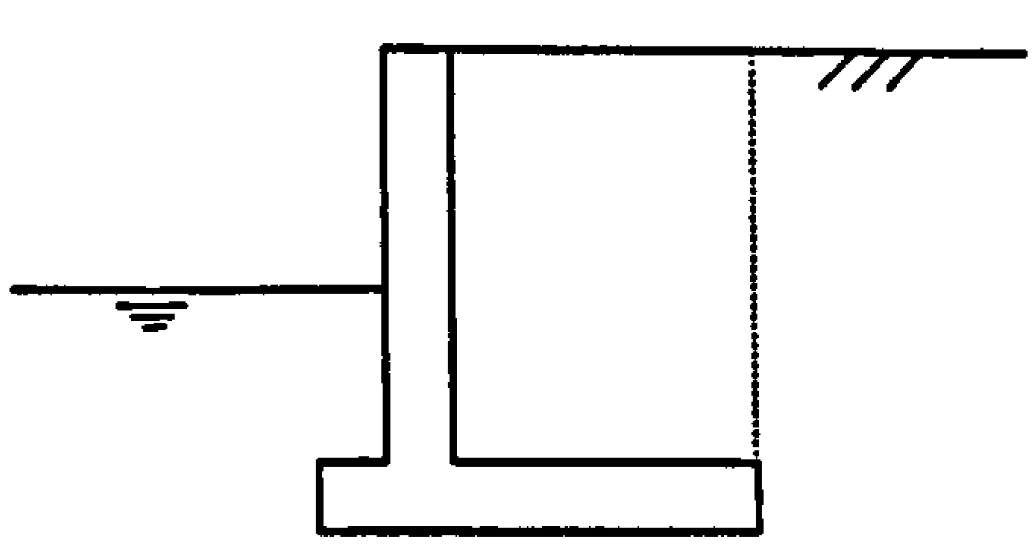


Figure D.18 A cantilever wall

D.8.4. Gabion walls

These are covered in Section D.5.3.

D.8.5. Sheet piles

These walls are only used when the height of the retained structure is relatively small, often they are for temporary support only. Failure is by rotation near the base of the wall and occurs when the surcharge, active pressures and hydrostatic forces exceed the restraining forces.

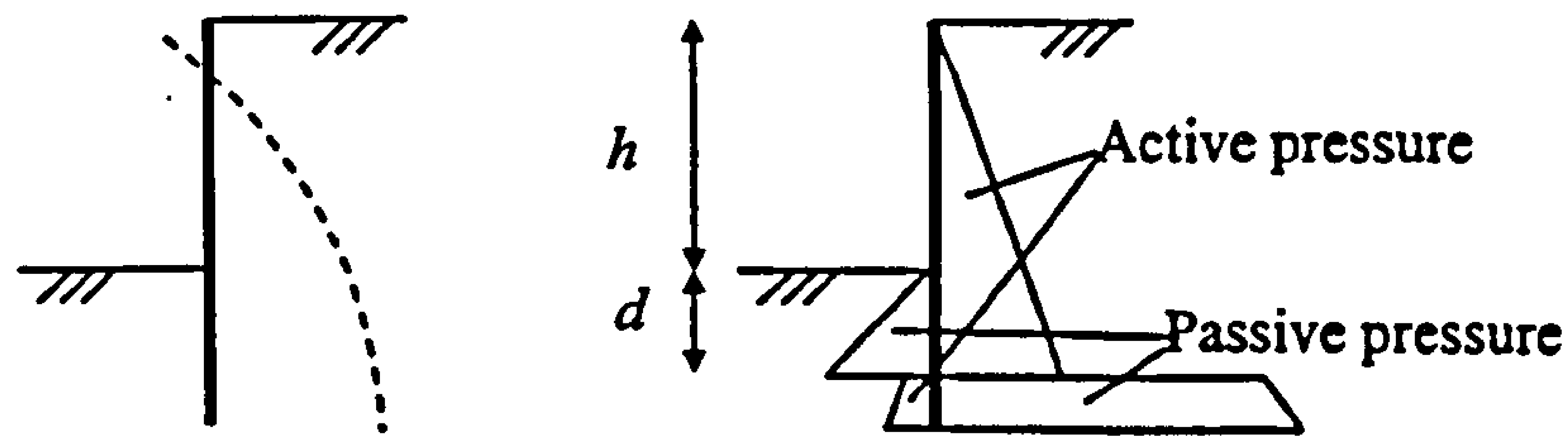


Figure D.19 Failure of a non-anchored sheet pile and resulting earth pressures

The depth, *d*, is determined by equating moments about the point of rotation to zero. A factor of safety is usually applied.

A sheet pile can receive additional support from a row of props or anchors behind the wall. These are normally steel cables or rods anchored in the soil some distance behind the wall. Plate anchors need to be far enough to ensure that the passive resistance mobilised by the anchor does not encroach on the active resistance of the wall. Ground anchors resist by creating skin friction between themselves and the soil.

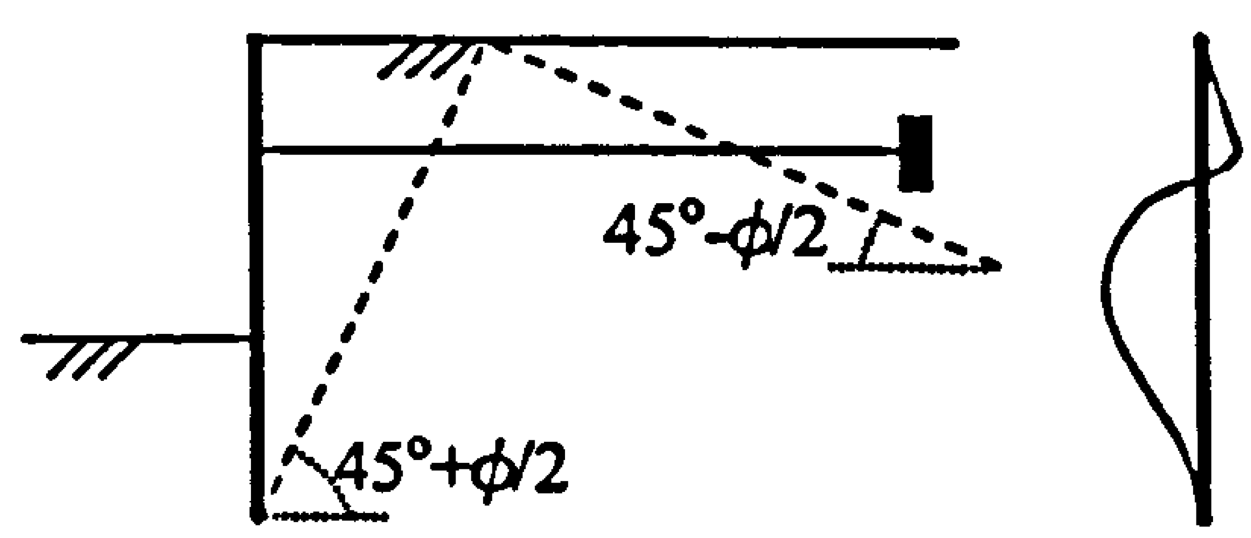


Figure D.20 Anchored sheet pile: Horizontal tie with a plot of the corresponding bending moment in the sheet pile

Anchored sheet piles can fail by:

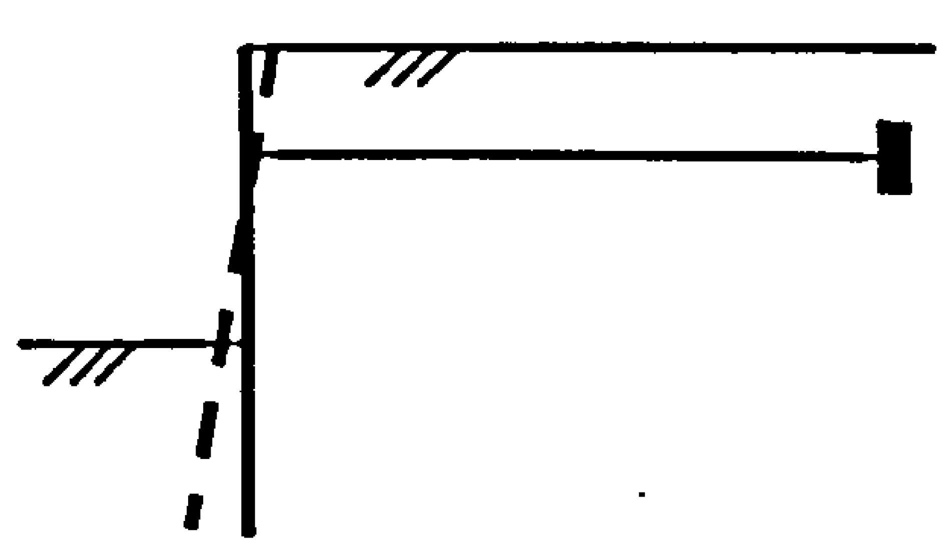


Figure D.21 Anchored sheet pile: Rotation at tie

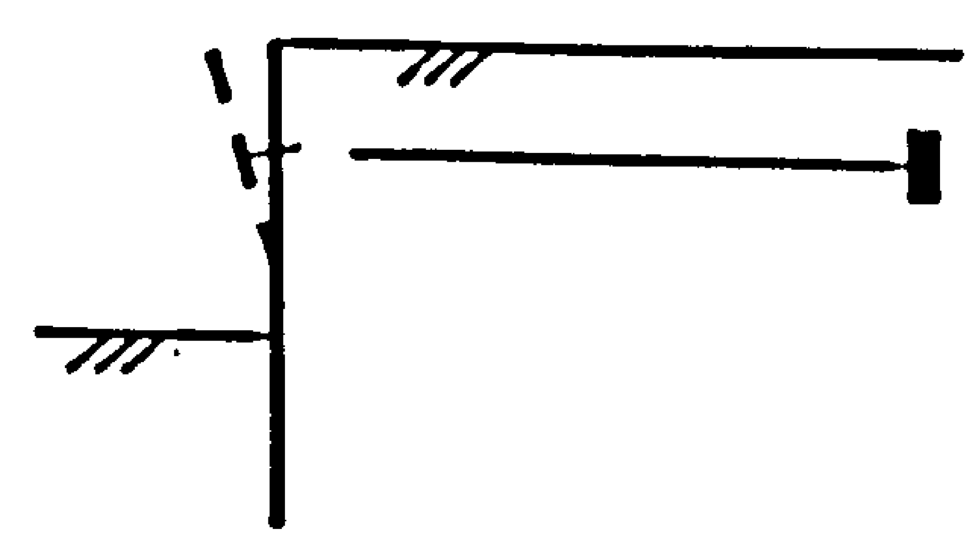


Figure D.22 Anchored sheet pile: Rupture of tie

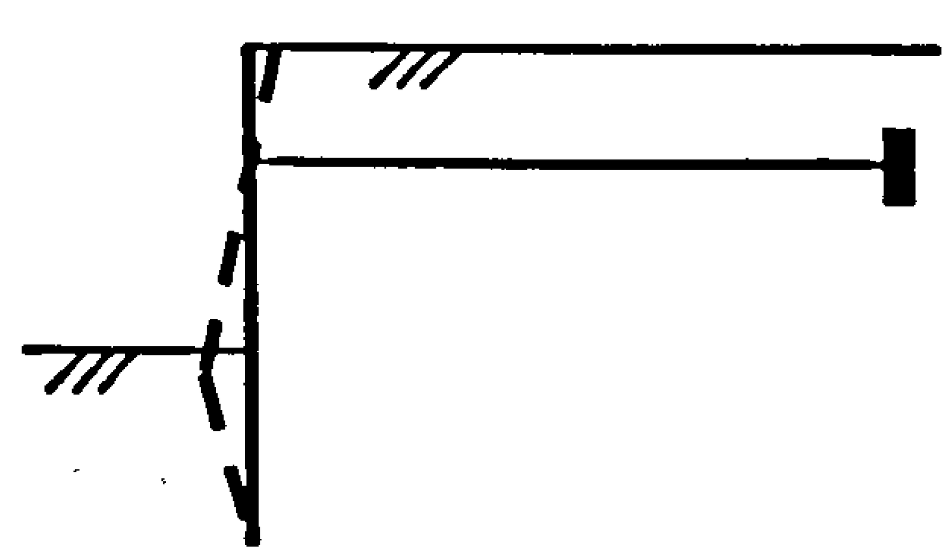


Figure D.23 Anchored sheet pile: Overstressing

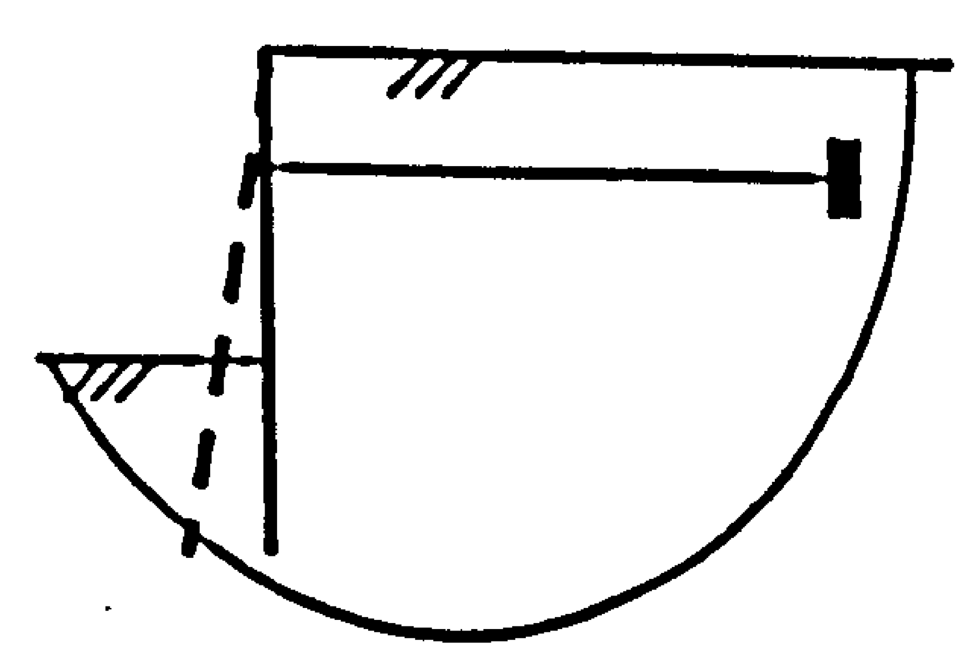


Figure D.24 Anchored sheet pile: Rotational slip

To design against tie rod failure, the tie rod force per unit length, *T*, is determined using:

$$Ts = \frac{\gamma d_a^2 l}{2F}(K_p - K_a) \tag{D.43}$$

where s is the rod spacing, F the factor of safety, l the rod length, d_a the depth from the surface to the bottom of the anchor. Proneness to failure by rotational slip is assessed using a standard slip circle analysis such as that described in Section D.7.3. Design against overstressing of the sheet pile is done by calculating the maximum bending moment in the wall and selecting an appropriate moment capacity of the sheet pile from design guides (such as those published by Corus Group, 2001). To design against rotation about the tie, the depth of embedment, d, is determined from the condition that the factored moments about the tie equate to zero. Another important consideration in sheet pile retaining structure design is the long and short term toe erosion. If toe erosion is underestimated, then the structure's stability could be compromised.

As with gabion frames, metal walls have to be protected suitably. Although unprotected sheet piles will normally have a life of at least 60 years. A rate of corrosion of 0.05mm/side/year for fluvial defences is assumed as a rule of thumb (Hemphill and Bramley, 1989) and this should be accounted for in long term design. However, Corus Group (2001) state that the variable quality of freshwater means corrosion rates are variable. Corrosion in marine environments can be up to 0.075mm/side/year and in certain cases 0.8mm/side/year has been observed.

D.8.6. Others

Reinforced earth

A composite structure of a concrete retaining face tied back into the body of the bank by long strips of webbing buried horizontally in the soil. The sole form of restraint is the resistive force between the webbing and the soil. Adequate drainage is imperative as high water pressures will reduce this resistance.

Retaining geotextiles

These can only be used at a limited height and not for high loadings or loose soil as the stability is dependent only on the stakes that support the material. The material must have long term stability under exposure to u-v light. In the long term, vegetation may grow through and around it to enhance appearance.

Rubber tyres

Two rows of rubber tyres with a filter layer behind them (and possibly an anchor) can be used. Although unattractive it is relatively inexpensive and provides a use for the large number of partially worn tyres that are discarded every year.

Crib walling

This wall type employs concrete or wooden beams to build up a lattice of interlocking members which are then backfilled. The lattice prevents minor slips and provides reinforcement to the bank. In the long term vegetation is established to help prevent soil being washed away.

D.9. NATURAL RIVER BANK PROTECTION

The design of natural defences is not really a quantitative process; experience and judgement are required in both the design and assessment of natural river protection. Another key issue is management; not only in the establishment of vegetation, but also in its long term state. Natural protection can be used to increase the resistance of a river bank to gradual erosion rather than mass failure.

D.9.1. River bank management

Plant selection

Key issues here are the local plant environment (would the addition of the wrong type of plant mean two plants competing), the suitability of the plant to the soil conditions and the role of the plant as a defence.

(1) Reeds
   (a) Used in the 'aquatic zone' of the bank to reduce wave and current energy
   (b) Their roots can be easily washed away and are therefore often protected by geotextiles and stone.

(2) Shrubs and trees (predominantly willow and poplar)
   (a) Can provide substantial reinforcement to the bank above and below waterline. .
   (b) Consideration must be provided for access to the waterway when planting.
   (c) Trees can help to stabilise the banks by reducing p.w.p. (although this effect is ended when the tree dies and the presence of the tree may then become detrimental to the stability of the bank) and their roots acting as reinforcement, but they can cause local scour if their roots become exposed (as discussed previously in this Appendix).

(3) Grass
   (a) Generally used above the normal water level
   (b) Reinforced grass uses geotextiles or concrete reinforcement to increase erosion resistance of the top soil.

(4) Timber
   (a) Cut timber can be driven in to the bank to reduce erosion
   (b) Thorn faggots are bundles of thorn branches that reduce erosion and increase drainage in the bank.
   (c) Toe boarding can be used to support banks and protect them from erosion.

Soil requirements

The correct soil conditions are required to achieve the intended plant growth.

Surface preparation

The intention is to prepare the ground to achieve the intended plant growth

(1) Production of a suitable seedbed

(2) Provide suitable drainage

(3) Relieve excessive subsoil compaction

(4) Improve soil structure

(5) Protection from immediate erosion whilst awaiting vegetation growth

Vegetation establishment

Method is chosen depending on plant type, location and soil type.

Zones of protection

As some plants respond well to wet soil and others prefer dry soil a policy of zoning can be instigated. Zoning allows permanently wet soil to benefit as much as possible from aquatic plants, and rarely flooded soil to benefit from other more suitable plantlife.

Long-term management

This is to ensure the vegetation is chosen and maintained to ensure long term objectives are met.

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Appendix E

Defence classification and fragility curves for use in a high level risk assessment

The classification builds previous work by the Environment Agency (1996). It is explicitly hierarchical, illustrating how, as more information is acquired, it is possible to define defence performance more precisely (see Figure E.3). Initially defences are classified into seven major types as shown by the third layer of the hierarchy in Figure E.1.

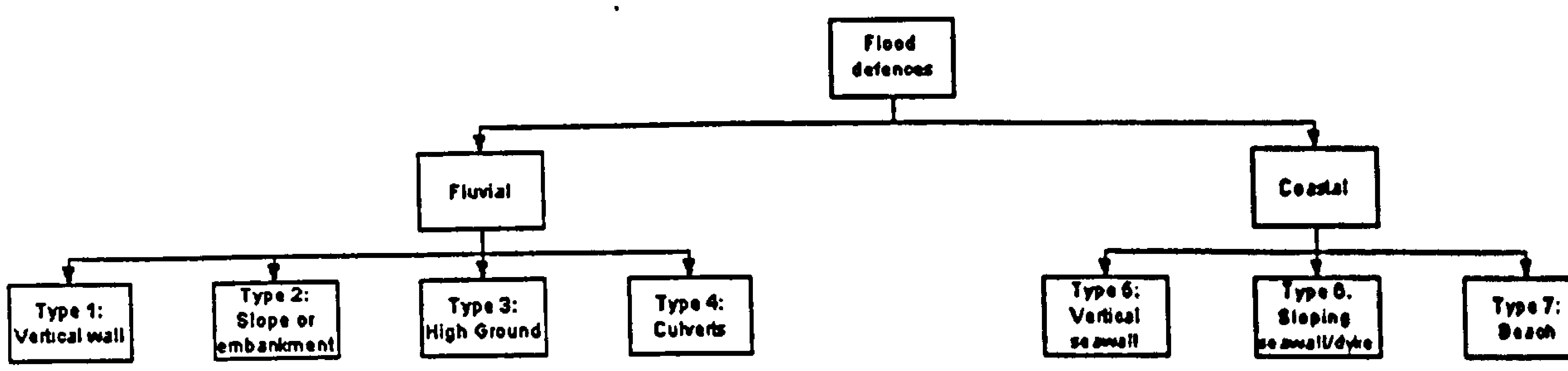


Figure E.1 Major classification groups of flood defences

The lower levels of the classification hierarchy for each of the defence types are shown in Figure E.3, Figure E.10, Figure E.19, Figure E.22, Figure E.29, Figure E.34 to the associated NFCDD codes are given in Table E.1 to Table E.6. The fragility curves for the different structure classifications are also displayed. These curves plot the mean value of the fragility. For the application fragility values of ±30% were used to reflect the uncertainty in the expert judgement used to define the curves.

A defence is usually composed of several components. For example a sea defence may have a foreshore, a frontslope, a crest and a backslope. All of these will have an influence on the proneness to failure of the defence. Only information on these components is stored and monitored by the Environment Agency as there is no generic defence classification field in the database (Environment Agency, 2001), however, the primary criterion for classification should be the aspect with the most influence on proneness to failure (see Appendix D for a thorough review of defence failure mechanisms). A classification routine, which was established by the author, was used to classify the defences into generic types based only on the available information describing their components.

The generic classification steps are as follows:

(1) Identify whether defence is coastal (including estuarial defences) or fluvial by checking the assets tidal flag in the NFCDD.

(2) Sub-divide into the seven major classes of defence as shown in Figure 5.5.

(3) Identify whether defence is 'wide' or 'narrow'. As in previous classification methodologies (Environment Agency, 1996) a wide defence has a crest level width of 10m or greater. If no crest level measurement is available a 'wide' defence is classified as not having a rear slope (NFCDD element code is FO).

(4) Ascertain degree of protection of defence based on whether they are protected on the front face, crest and rear face. Protection is assumed if the revetment material of the asset element is not turf or trees. Examples of defences classified by width and by degree of protection are shown in Figure E.2.

Note: For the purposes of implementing the algorithm, it is necessary to know whether there is an outward slope to perform step 3, but the width classification is at a higher level than the degree of protection classification.

(5) Sub-classify depending on material of front face protection (no classification based on crest or rear slope material).

(6) Identify any structures (*eg.* outfalls) within defence as this will effect the defence's fragility. This step was not implemented, see Section E.7 for more details.

(7) Finally, further classification can be made based on whether the channel (in the case of fluvial defences) is lined or unlined or (in the case of sea defences) whether the defence is tidal or coastal. This step was not implemented; due to data limitations it was not possible to discriminate between tidal and coastal defences, and, experts were unable to differentiate between the strength of structures with lined and unlined channels.

Initially, it may only be possible to construct fragility curves at relatively high levels of the classification. As more information becomes available, the fragility curves at lower levels can start to be populated. This increase in data will also allow the bounds on the fragility curves to be narrowed as demonstrated in Figure 5.7. Further refinement will be possible in more detailed analysis when the dimensions of the defence become available.

The following sections link the classification methodology with the codes describing asset element types, sub-types, material and revetment used to populate the NFCDD. The NFCDD codes are defined in Section E.8. The following notation is used in all the fragility curve graphs:

- P(OT) is the fragility curve for the conditional probability of overtopping
- P(B) is the fragility curve for the conditional probability of breaching
- CGx is the condition grade of the defence (where x=1 corresponds to "very good" and x=5 corresponds to "very poor" condition.
- FP means the defence only has front protection

- CP means the defence has front and crest protection, and,
- RP means the defence has front, crest and rear protection.

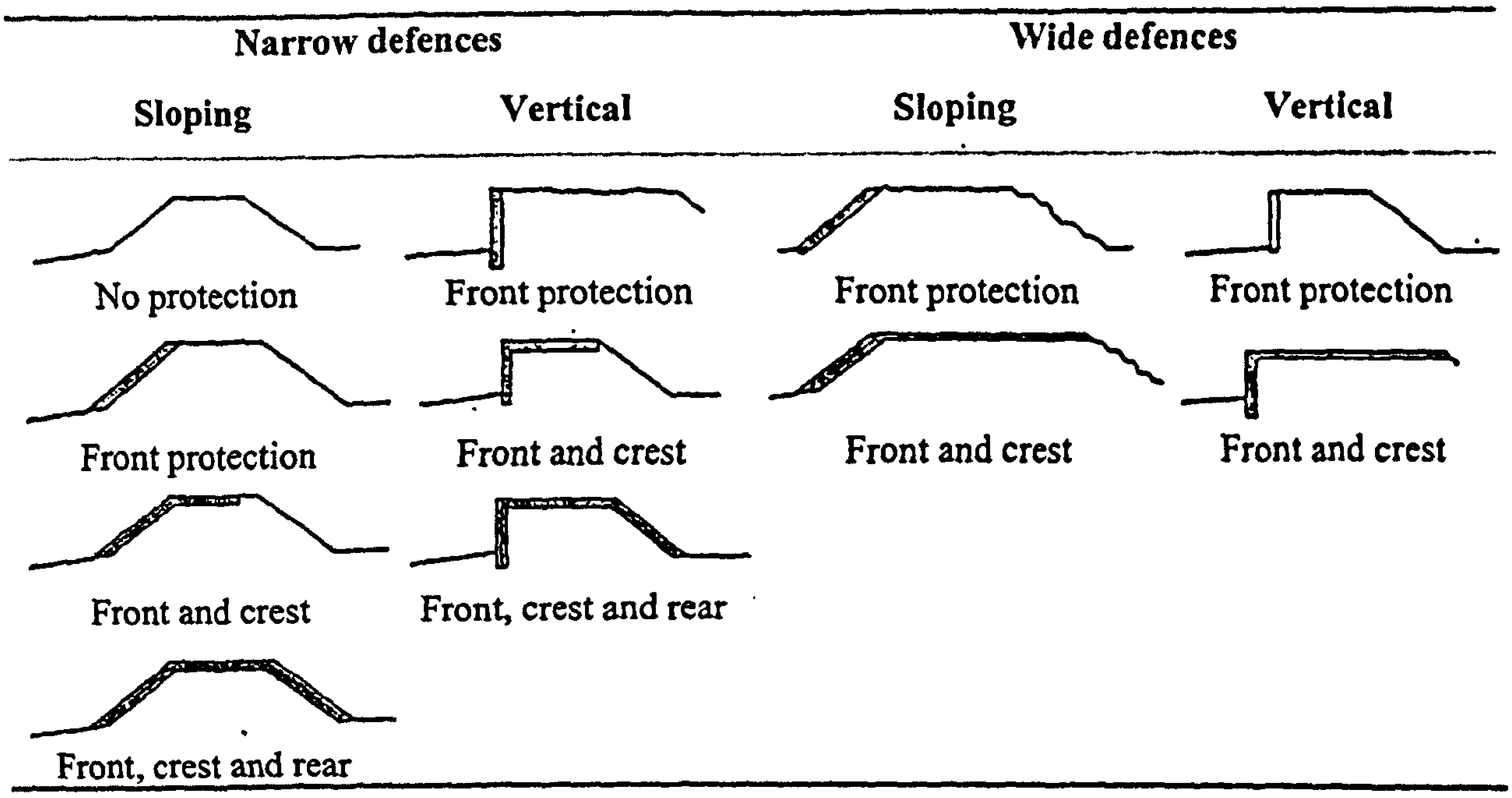
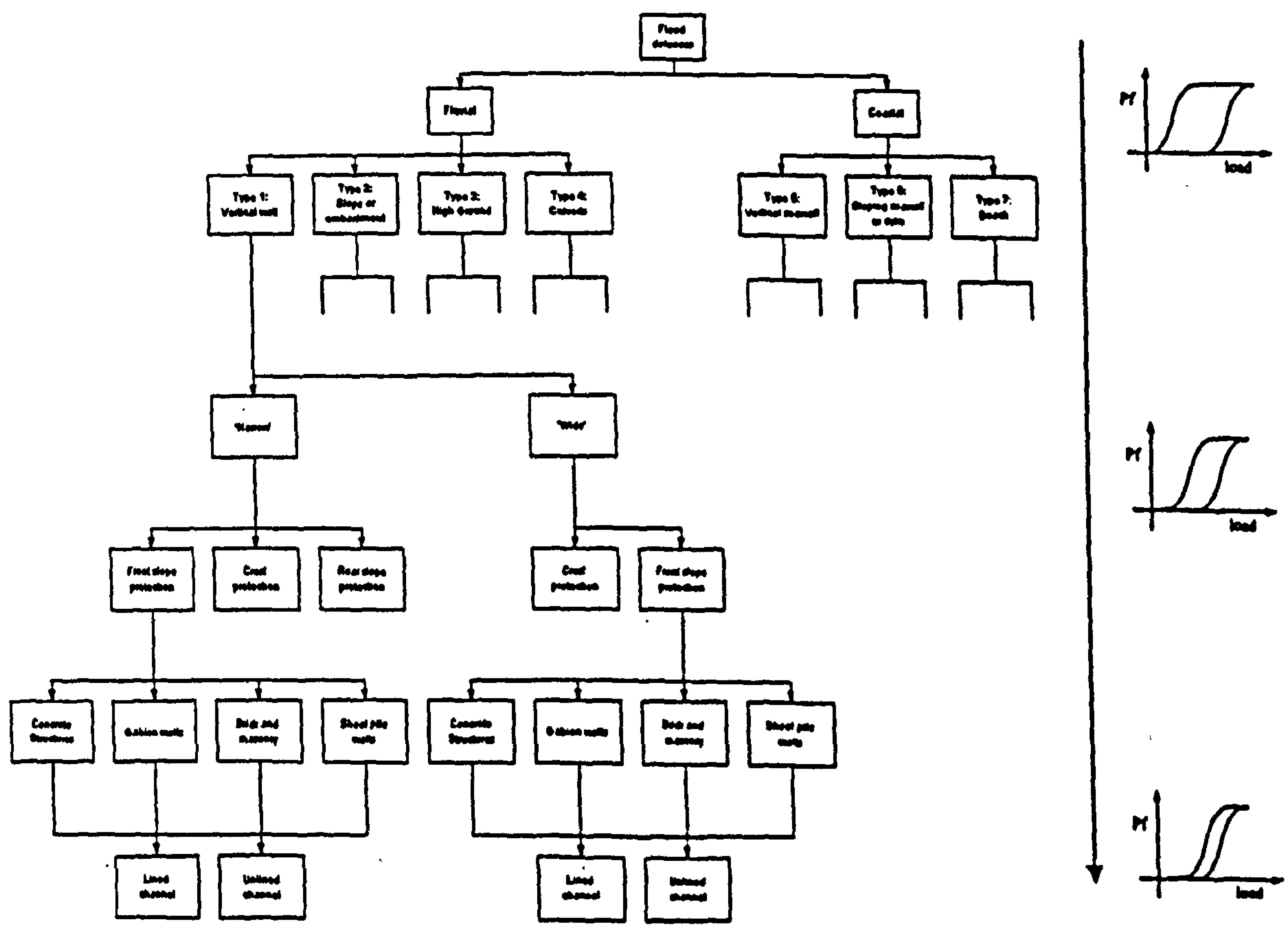


Figure E.2 Example of defence classification based on defence width and crest and rear slope protection (Environment Agency, 1996)

E.1. TYPE 1: VERTICAL RIVER WALLS



Note: Only front protection is classified further by material type.

Figure E.3 Detailed classification of vertical fluvial defences showing how more sophisticated classification enables the bounds of the associated fragility curve to be narrowed

Table E.1 Classification description and associated NFCDD codes for vertical fluvial defences

Classification	Relevant NFCDD codes Type	Sub-type	Material	Revetment
Vertical wall				
Concrete structures	Channel: CB	L/N/R	C/D/F/G/H/M/O/R/S	-
(reinforced or gravity)	Defence: CS/FI/BE/FC/FO	H/W	C/D/Q/R	-
Gabion walls	Channel: CB	L/N/R	C/D/F/G/H/M/O/R/S	-
	Defence: CS/FI/BE/FC/FO	H/W	G	-
Brick and masonry structures	Channel: CB	L/N/R	C/D/F/G/H/M/O/R/S	-
	Defence: CS/FI/BE/FC/FO	H/W	M	-
Sheet pile walls	Channel: CB	L/N/R	C/D/F/G/H/M/O/R/S	-
	Defence: CS/FI/BE/FC/FO	H/W	P	-

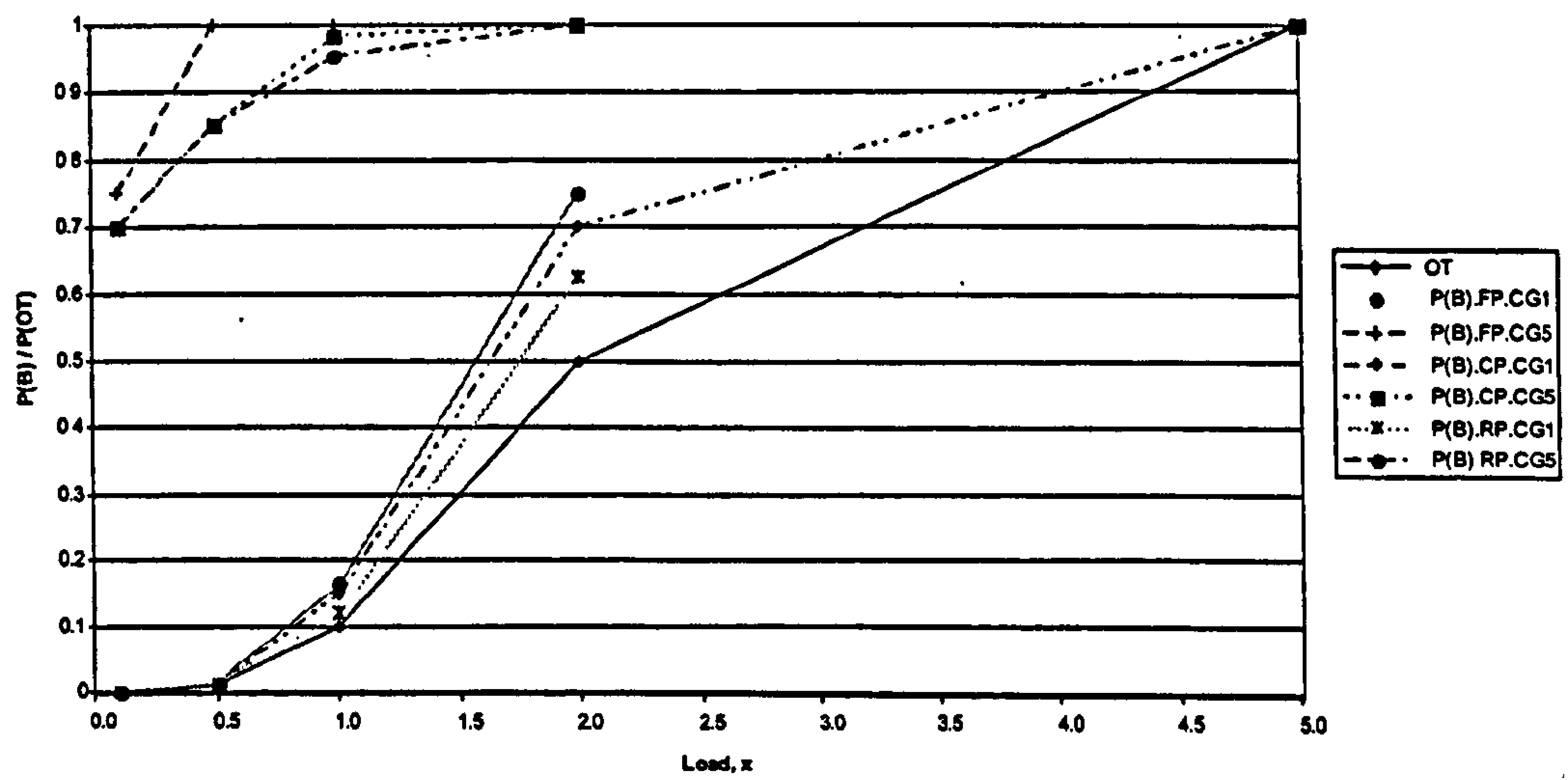


Figure E.4 Fragility curves for vertical fluvial defence: narrow, front protected by gabions

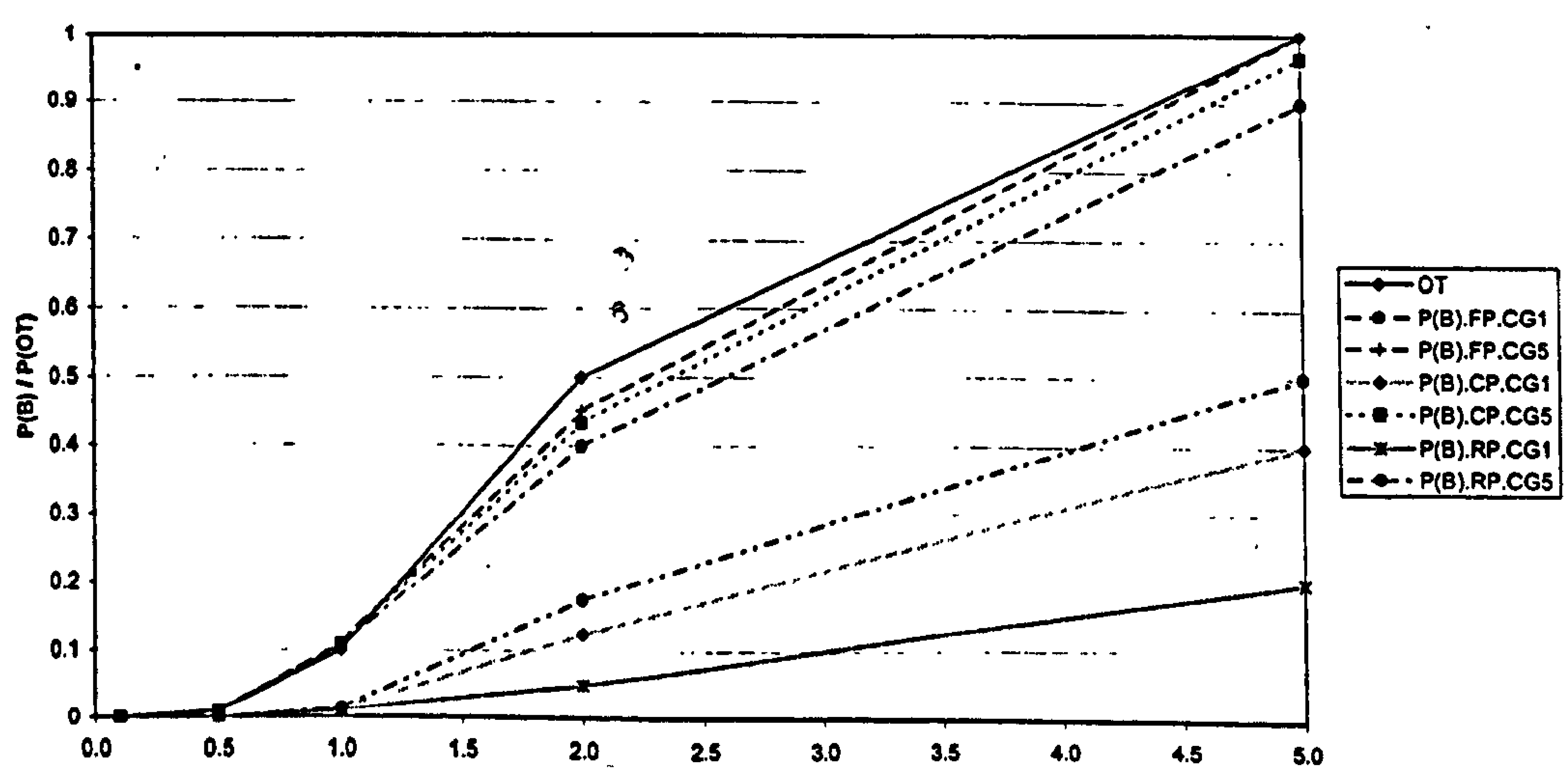


Figure E.5 Fragility curves for vertical fluvial defence: narrow, front protected by concrete or bricks and masonry

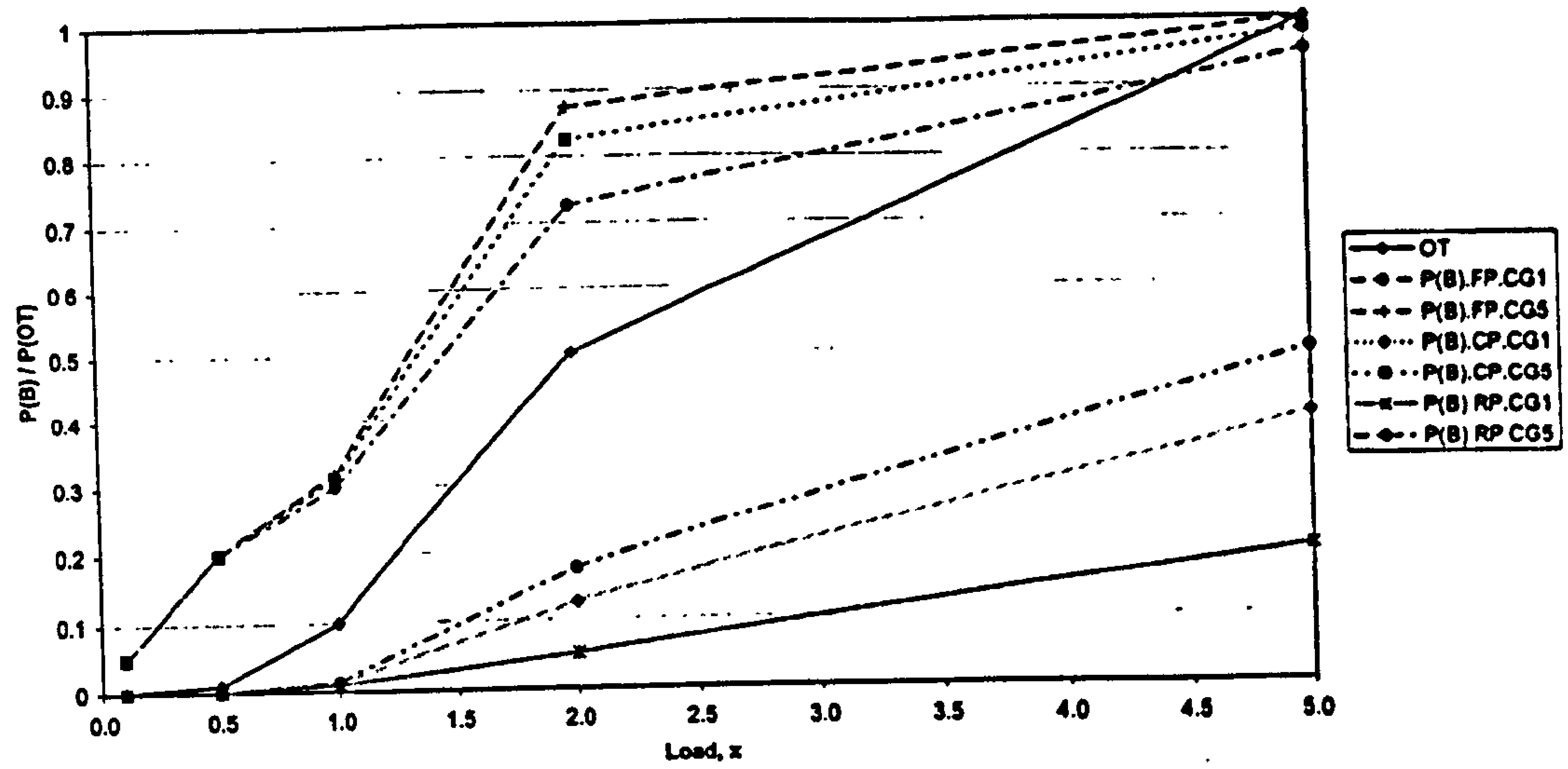


Figure E.6 Fragility curves for vertical fluvial defence: narrow, front protected by sheet piles

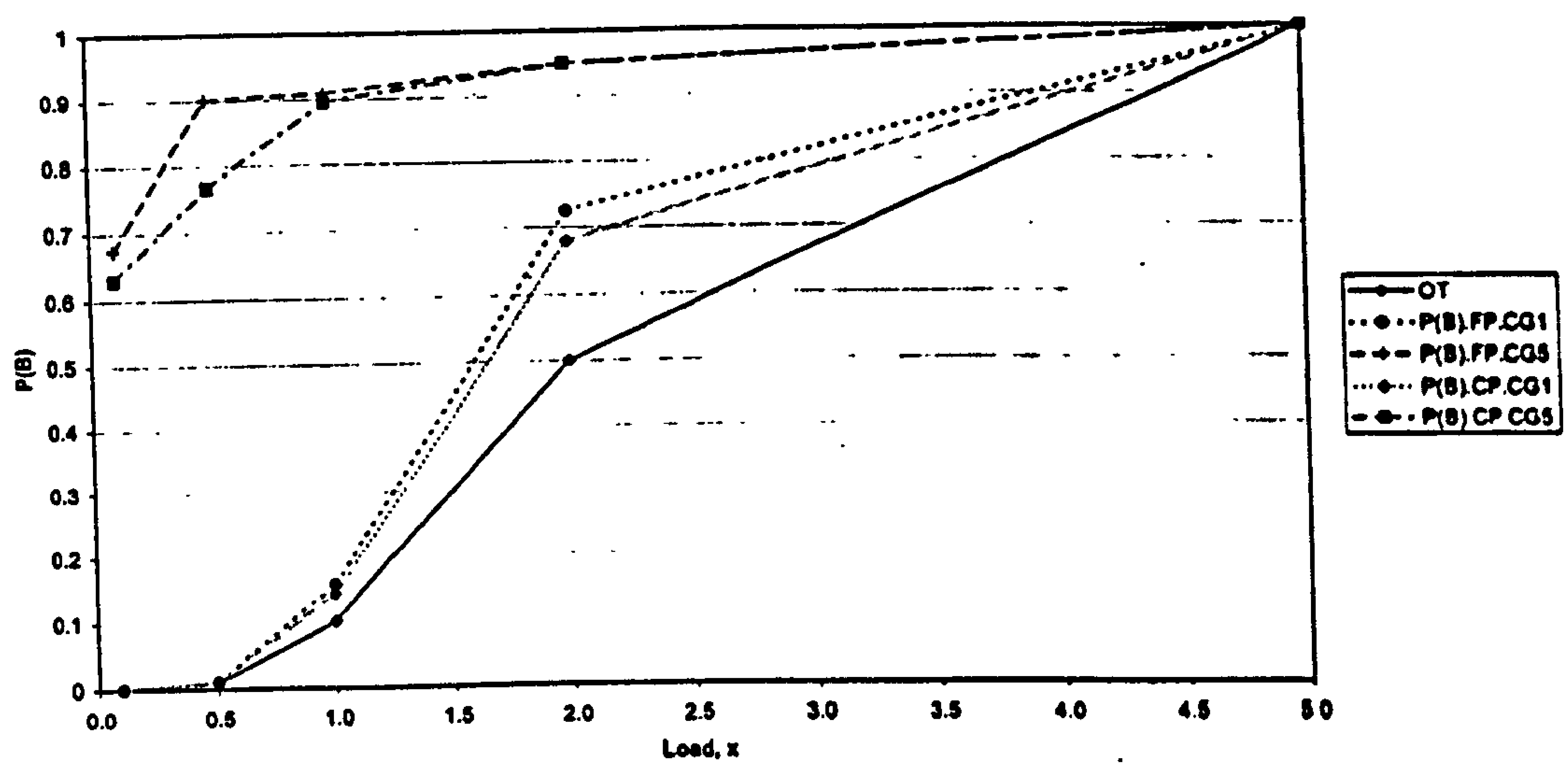


Figure E.7 Fragility curves for vertical fluvial defence: wide, front protected by gabions

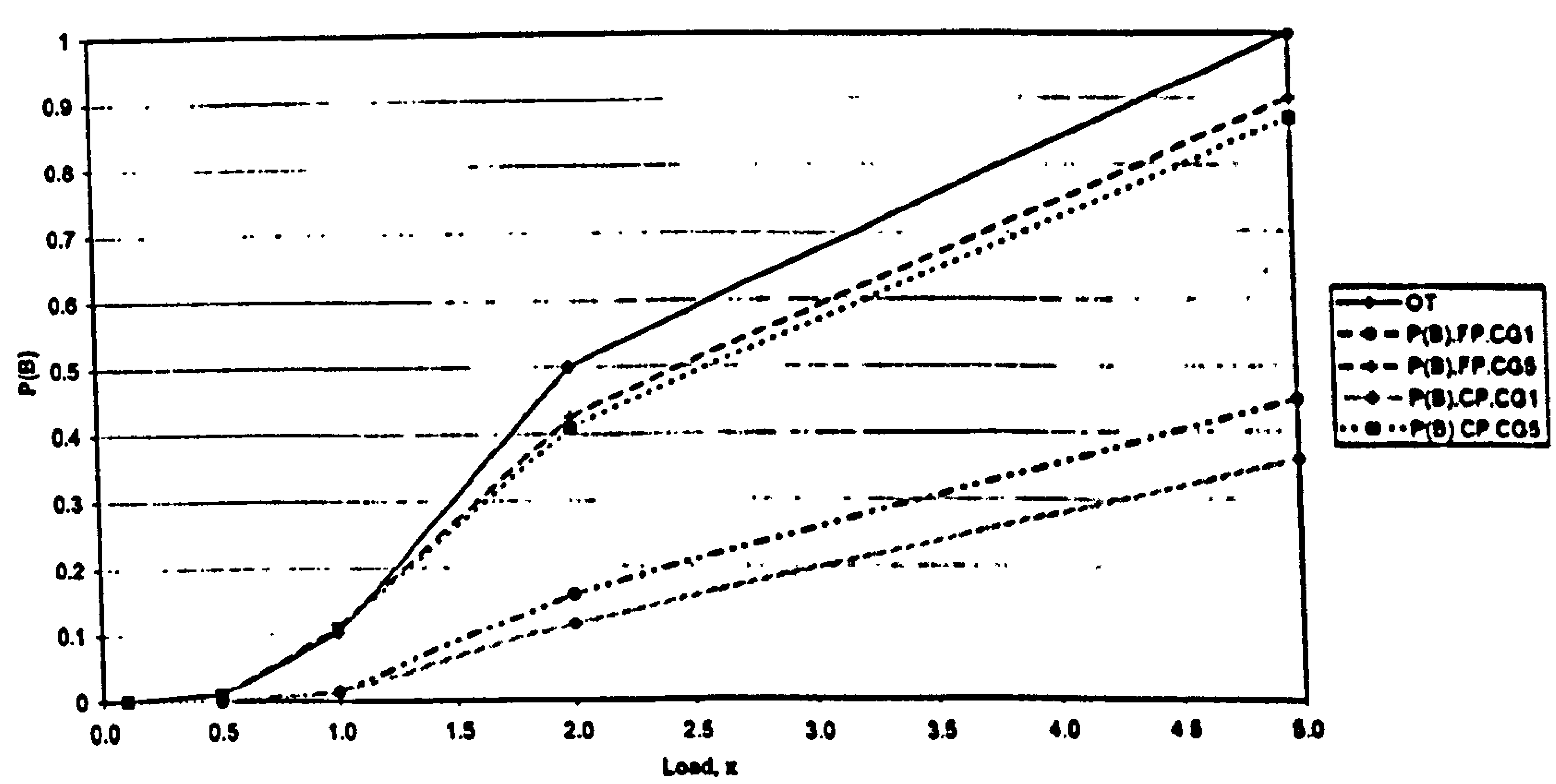


Figure E.8 Fragility curves for vertical fluvial defence: wide, front protected by concrete or bricks and masonry

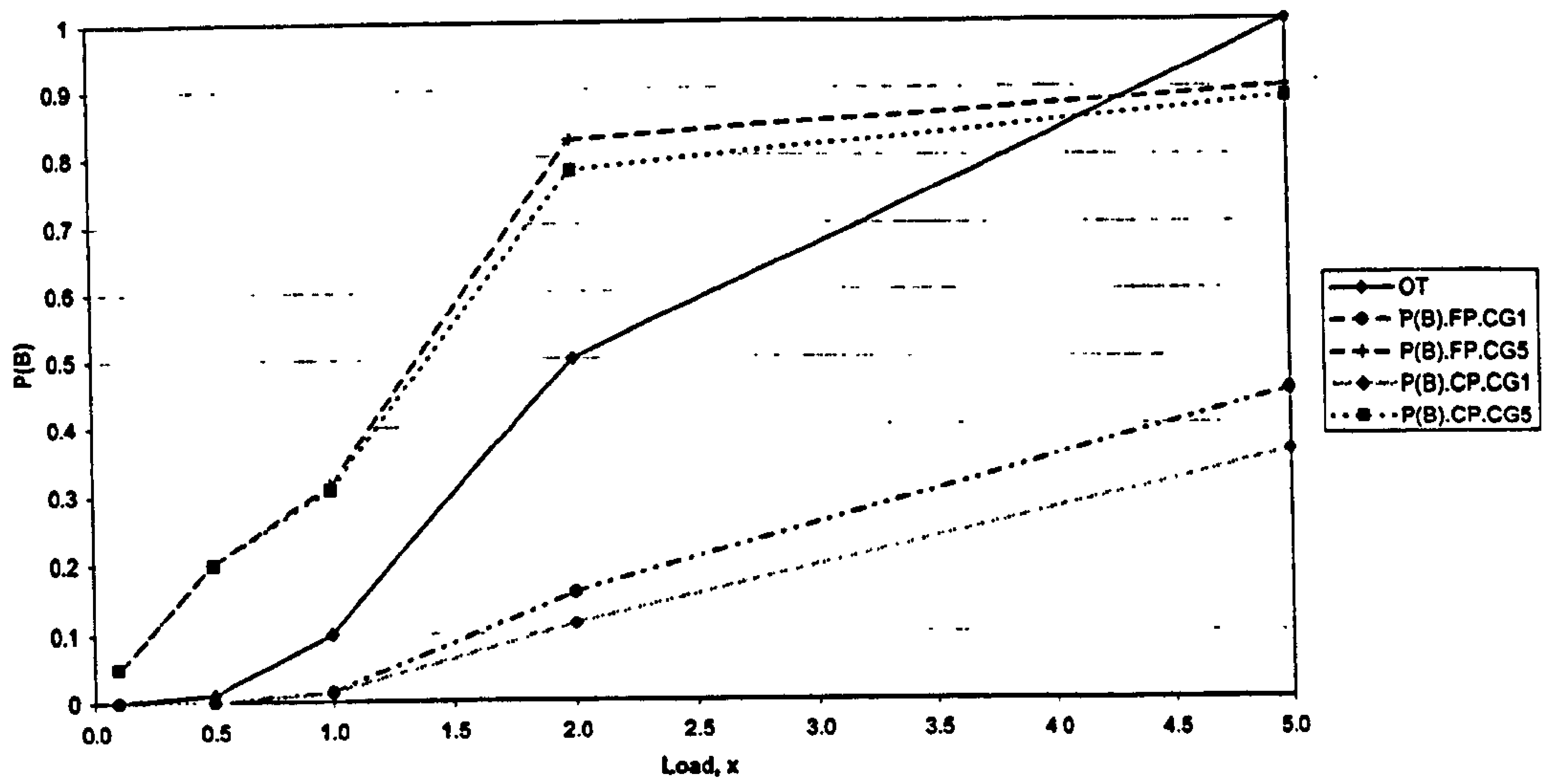
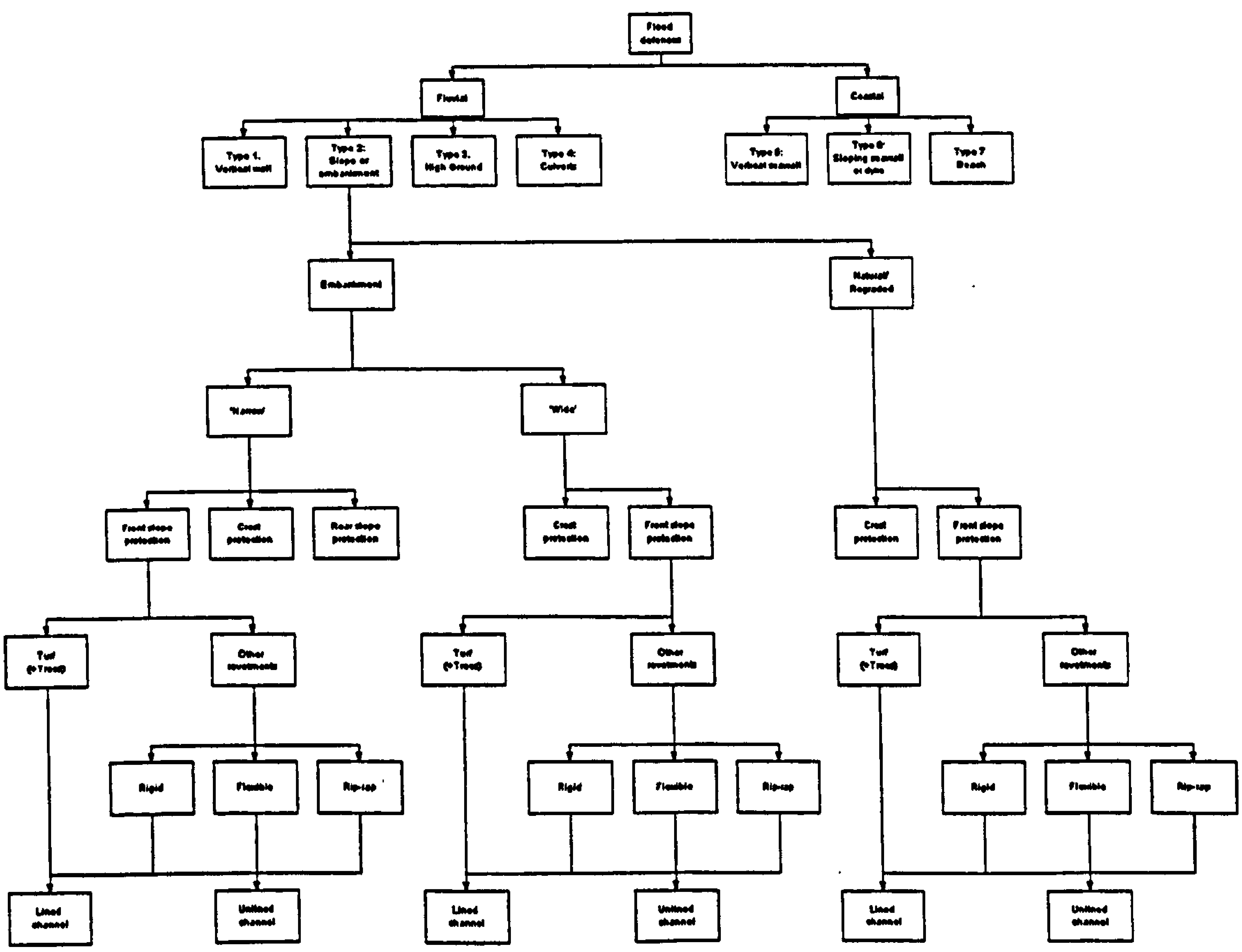


Figure E.9 Fragility curves for vertical fluvial defence: wide, front protected by sheet piles

E.2. TYPE 2: SLOPES OR EMBANKMENTS



Note: Natural and regarded banks are assumed to be 'wide' unless a crest width is specified

Note: Rigid revetments include concrete slabs and flexible revetments include asphalt, concrete blockwork and pitched stone.

Note: No further classification of Type 3 (High Ground) is necessary.

Figure E.10 Detailed classification of fluvial slopes or embankments

Table E.2 Classification description and associated NFCDD codes for sloping fluvial defences

Classification	Type	Sub-type	Material	Revetment
Slope or embankment				
Embankment	Channel: CB/CS	L/N/R/W	C/D/F/G/H/K/M/N/O/R/S/W (K/N/W/Z)	-
	Defence: FI/BE/FC/FO/(DO)	B (VE/GE)	E/V/ (K/N/W/Z)	Flexible: A/B/F/S Rigid: H/J/Y Rip-rap: U/W Flex/Rigid: O Other: Z
Regraded	Channel: CB/CS	L/N/R/W	C/D/F/G/H/K/M/N/O/R/S/W (K/N/W/Z)	-
	Defence: FI/BE/FC/FO/(DO)	R (VE/GE)	E/V/ (K/N/W/Z)	Flexible: A/B/F/S Rigid: H/J/Y Rip-rap: U/W Flex/Rigid: O Other: Z
Natural	Channel: CB/CS	L/N/R/W	C/D/F/G/H/K/M/N/O/R/S/W (K/N/W/Z)	-
	Defence: FI/BE/FC/FO/(DO)	*N* (VE)	E/V/ (K/N/W/Z)	Flexible: A/B/F/S Rigid: H/J/Y Rip-rap: U/W Flex/Rigid: O Other: Z

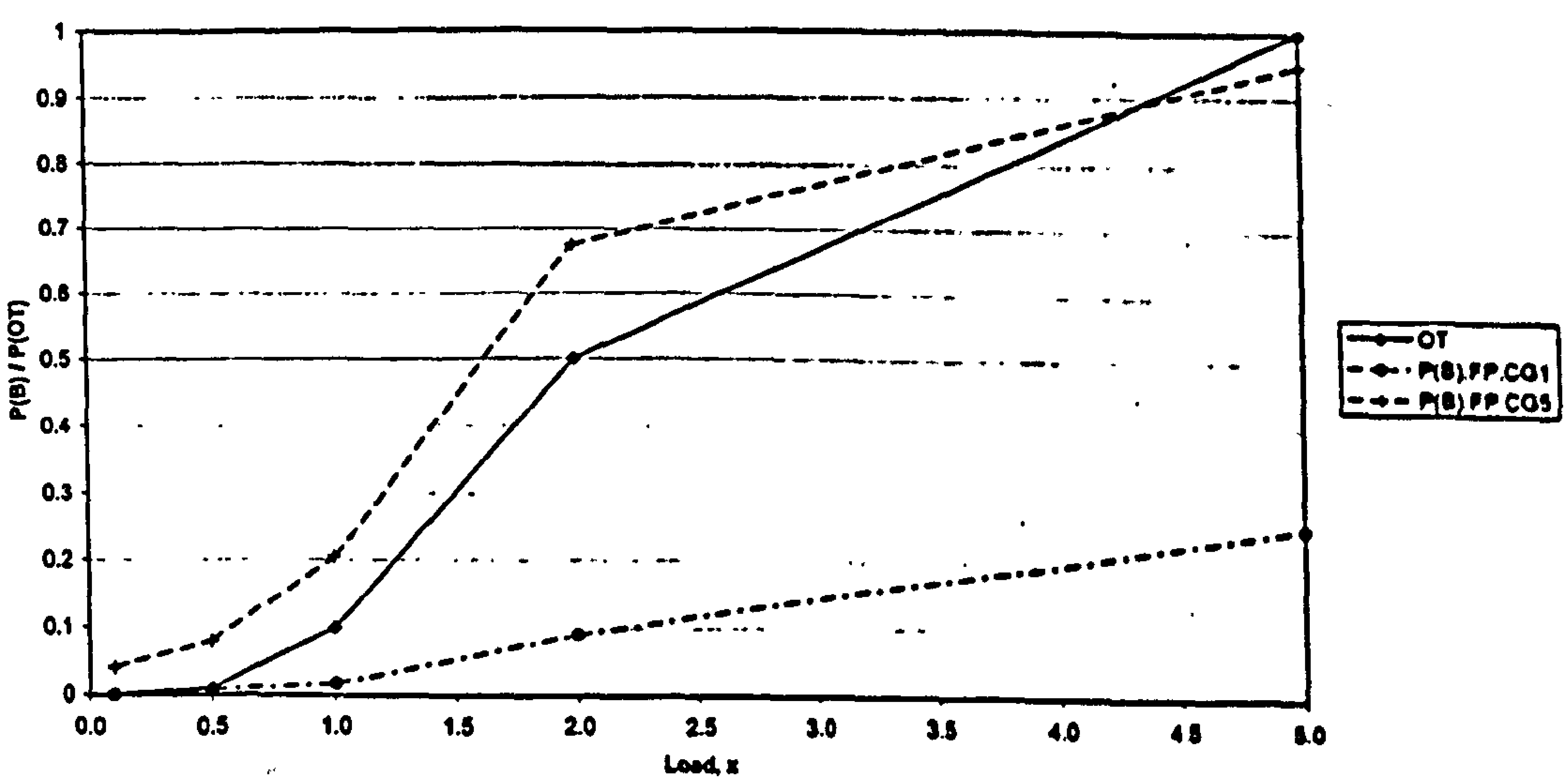


Figure E.11 Fragility curves for sloping fluvial defence: narrow, front protected is natural

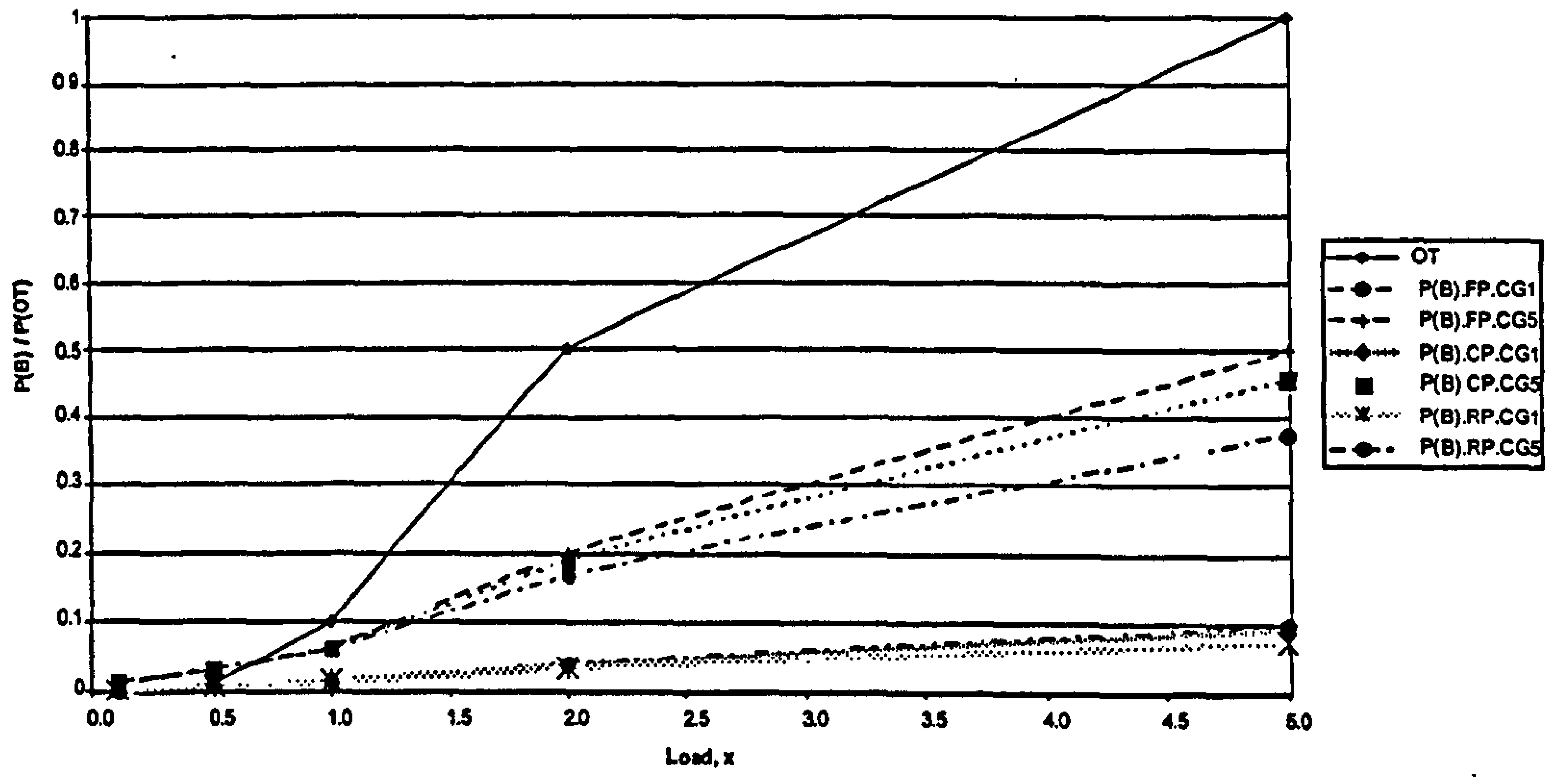


Figure E.12 Fragility curves for sloping fluvial defence: narrow, front protection is rigid

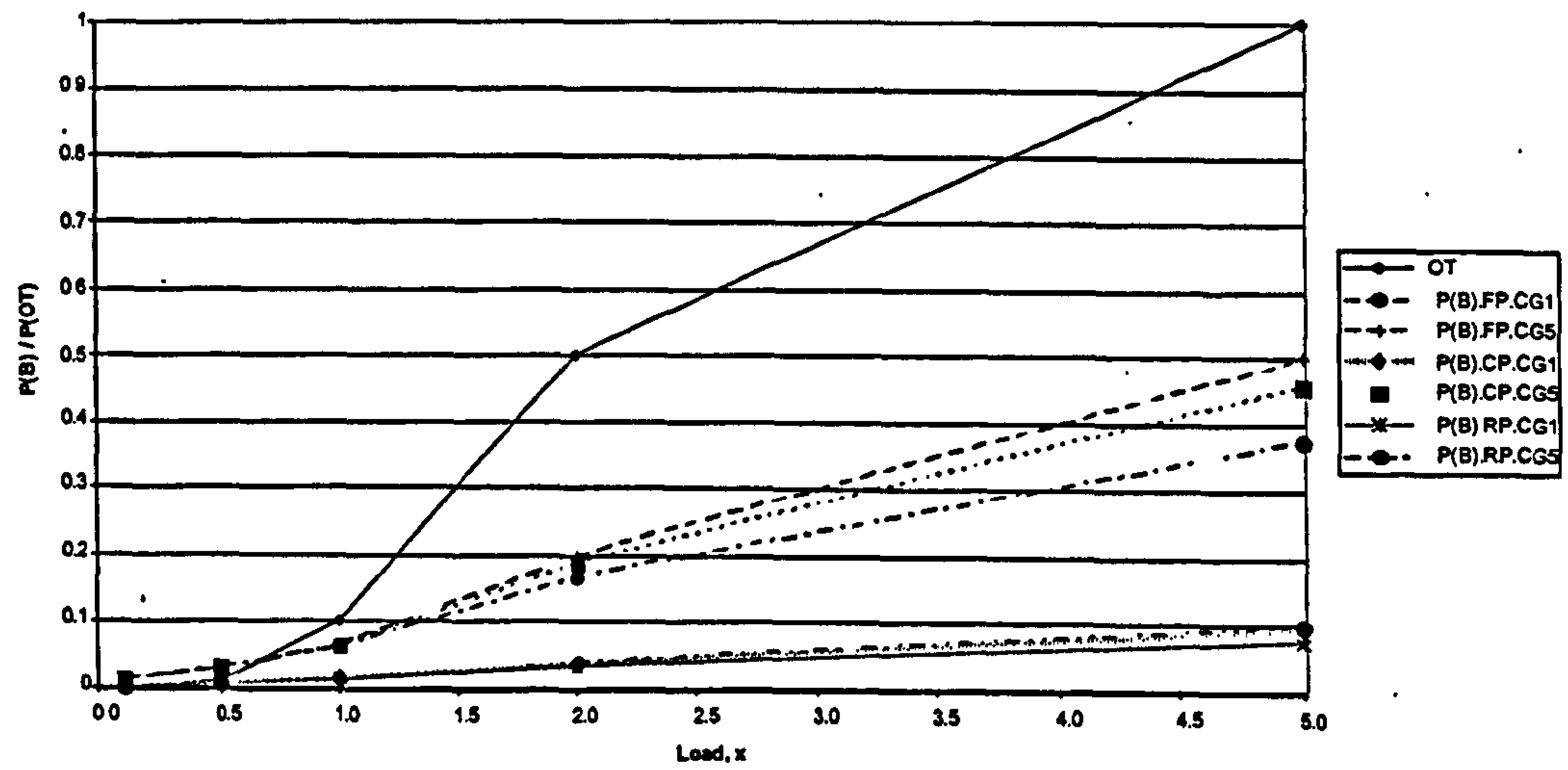


Figure E.13 Fragility curves for sloping fluvial defence: narrow, front protection is flexible

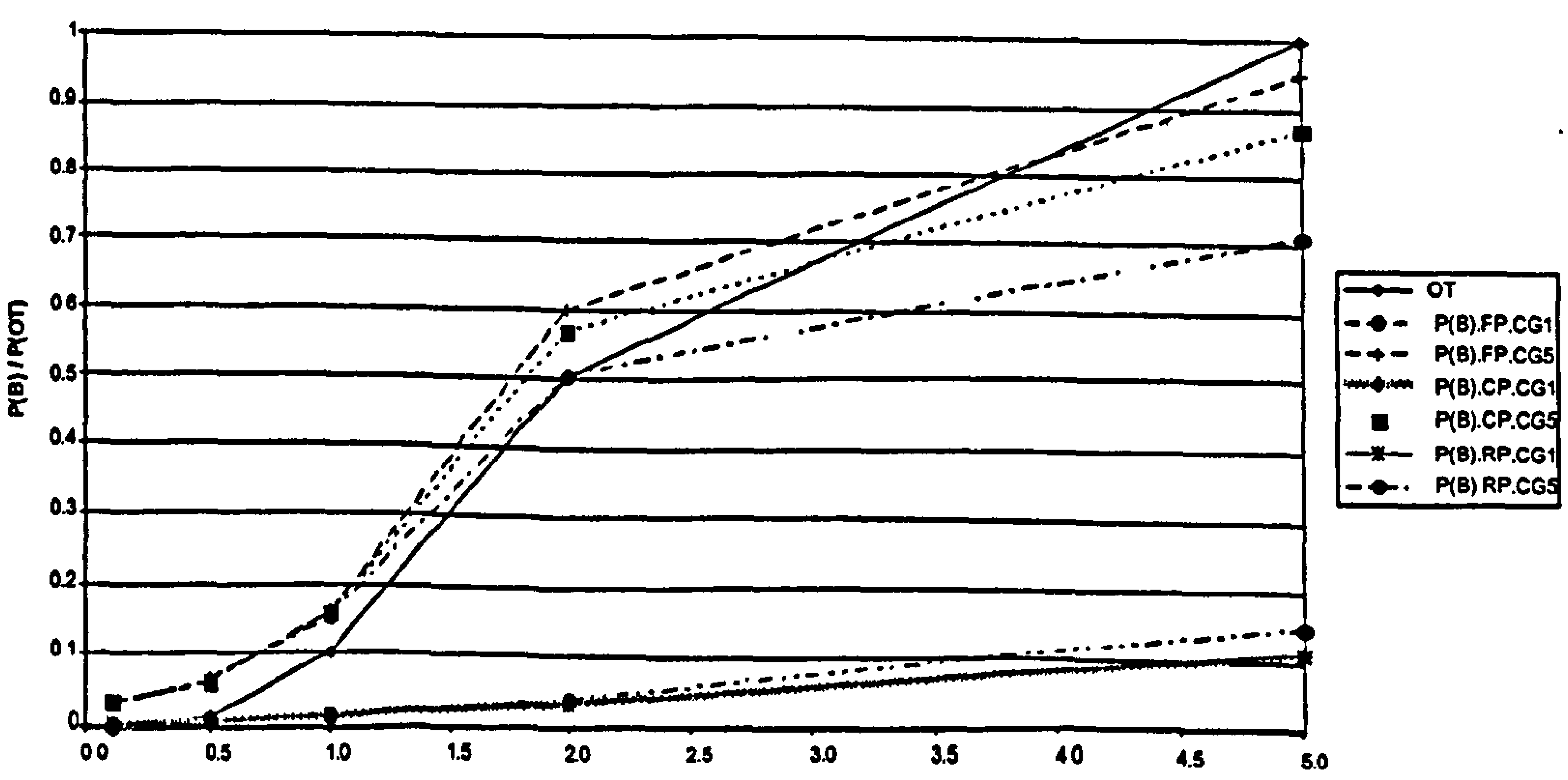


Figure E.14 Fragility curves for sloping fluvial defence: narrow, front protection is rip-rap

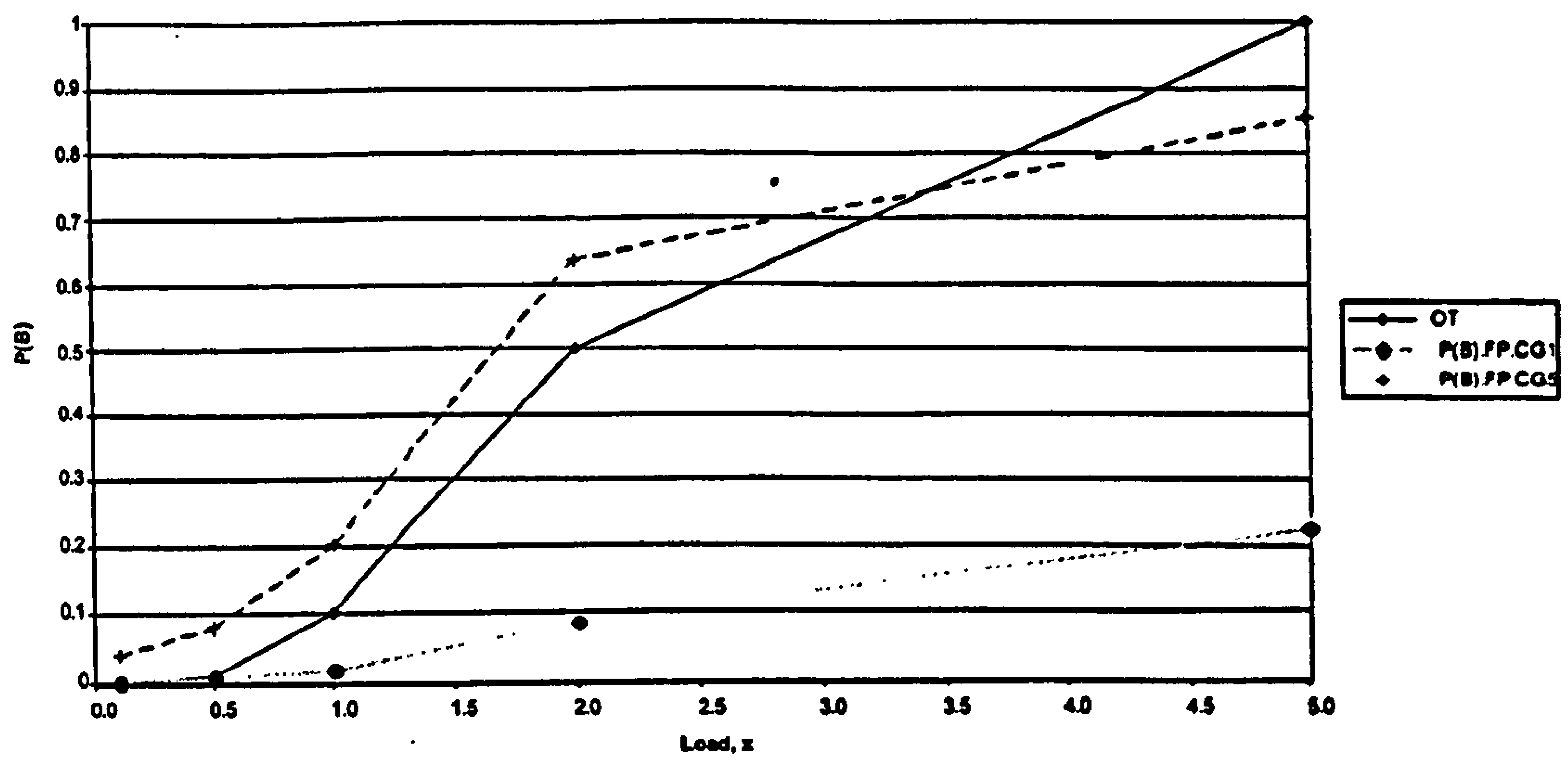


Figure E.15 Fragility curves for sloping fluvial defence: wide, front protection is natural

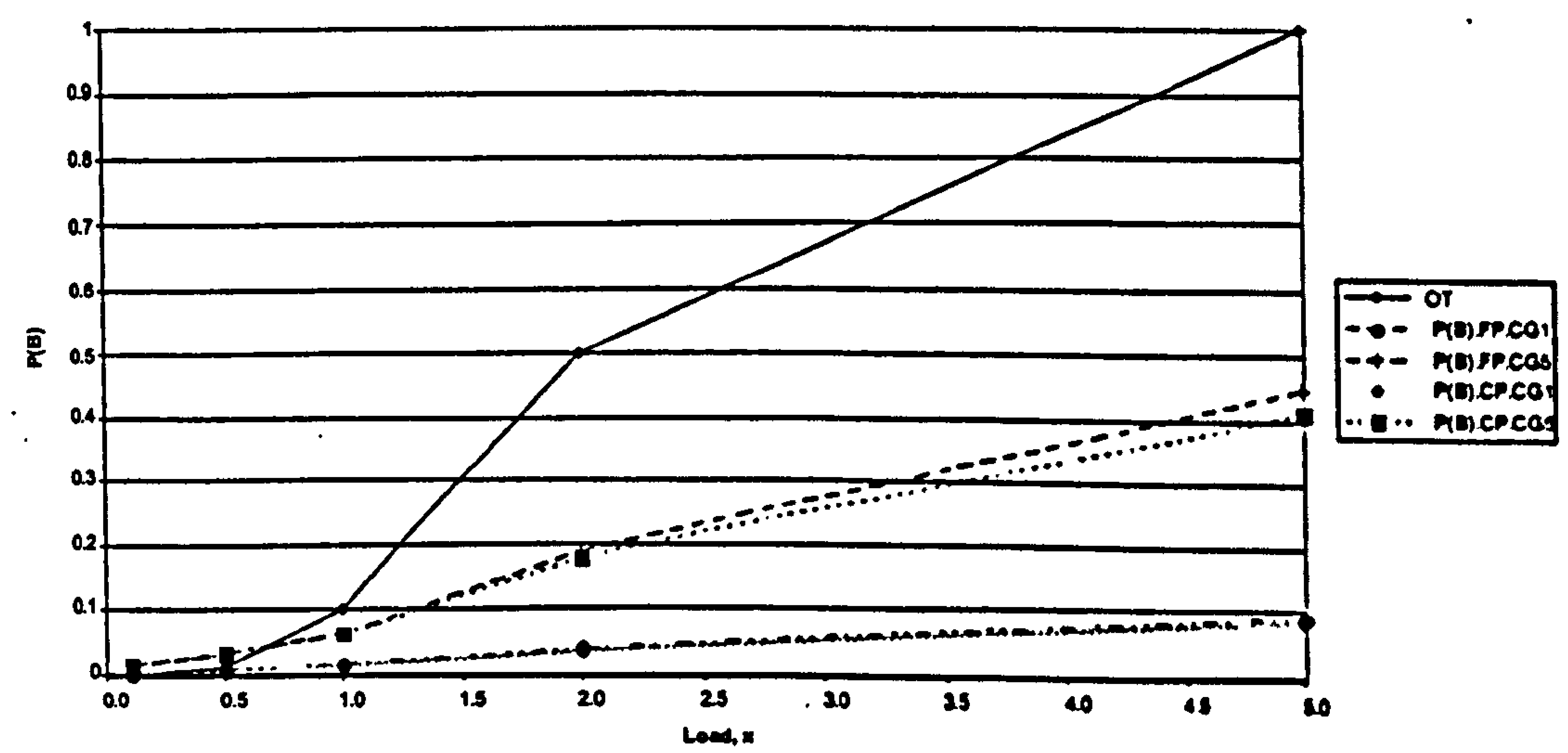


Figure E.16 Fragility curves for sloping fluvial defence: wide, front protection is rigid

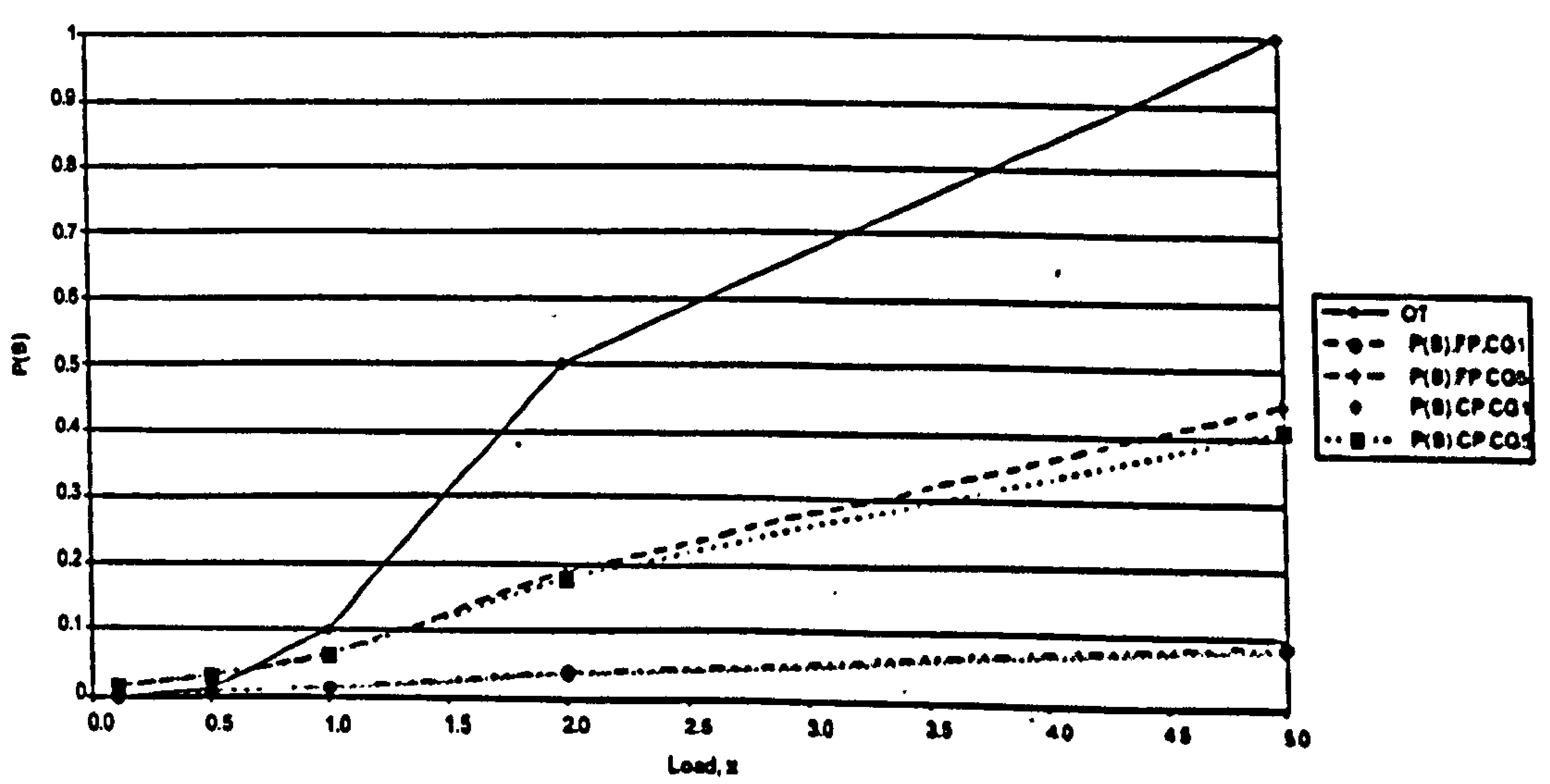


Figure E.17 Fragility curves for sloping fluvial defence: wide, front protection is flexible

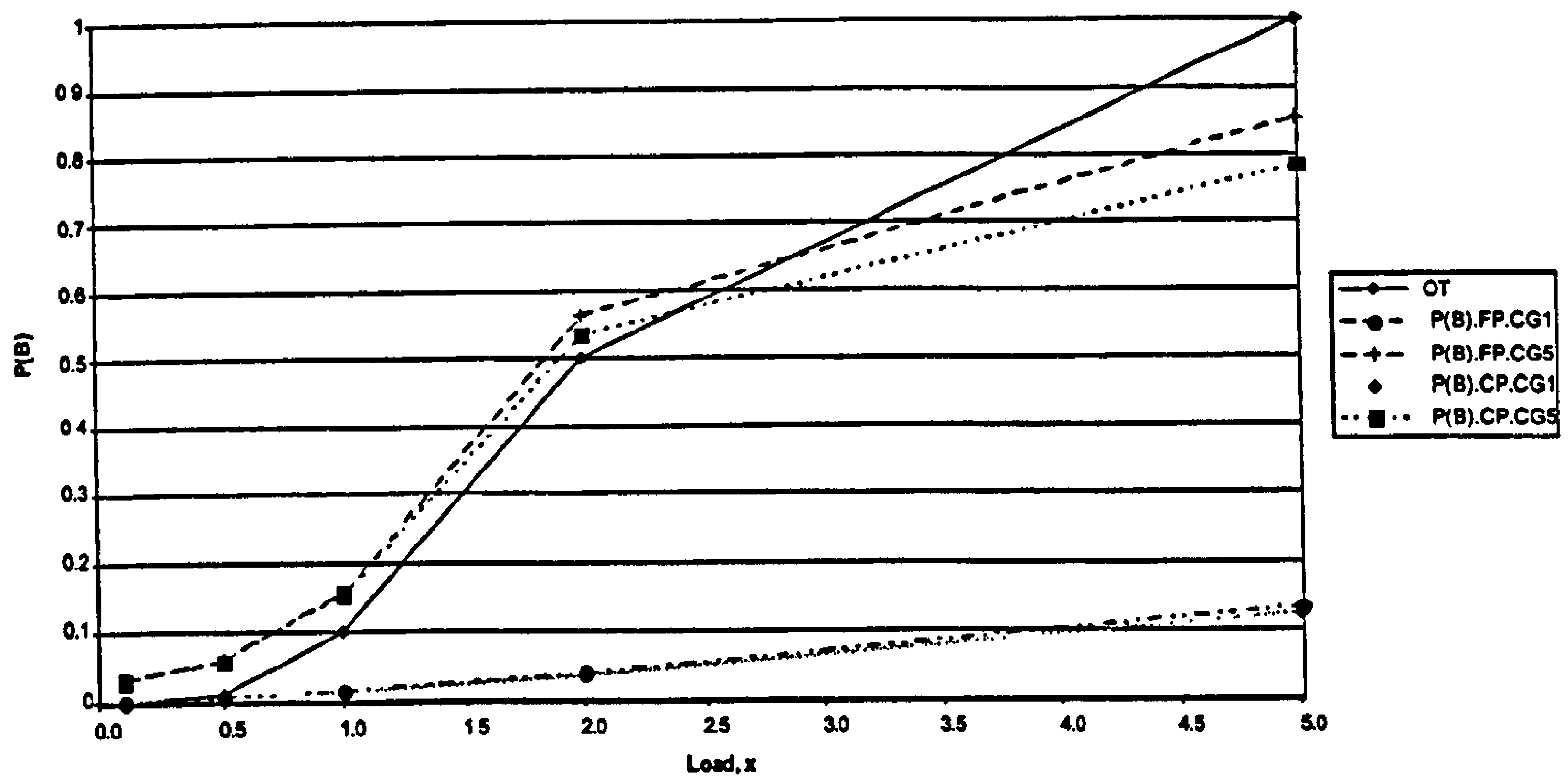


Figure E.18 Fragility curves for sloping fluvial defence: wide, front protection is rip-rap

E.3. TYPE 3 AND 4: HIGH GROUND AND CULVERTS

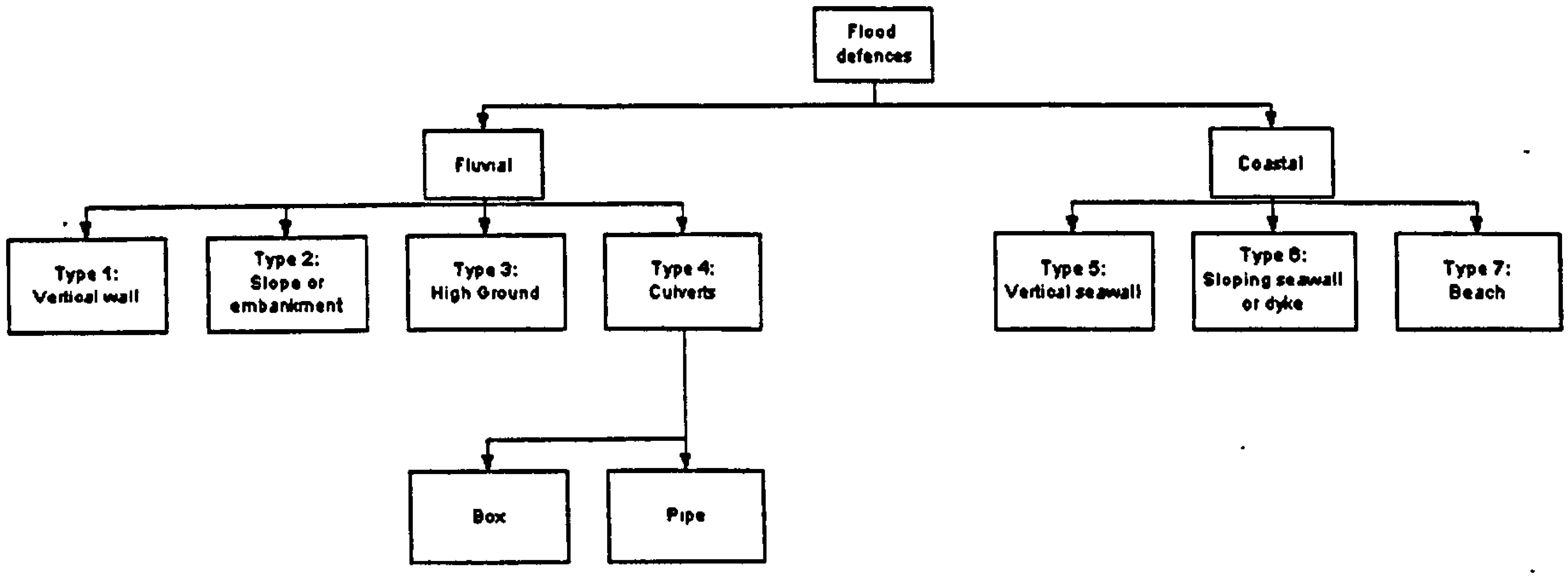


Figure E.19 Detailed classification of culverts

Table E.3 Classification description and associated NFCDD codes for high ground and culverts

Classification	Type	Sub-type	Material	Revetment
High Ground	HG	-	-	-
Culvert				
Box	CU	BC	A/B/C/D/F/L/M/O/P/Q/S/X	-
Pipe	CU	PI	A/B/C/D/F/L/M/O/P/Q/S/X	-

Note: VE and GE (vegetation and geotextile) are bracketed to demonstrate that they can be associated with element types, but not important in the classification process.

Note: CU (culvert) may have many associated elements (such as protection to the entrance and exit), but these are not relevant to the classification process.

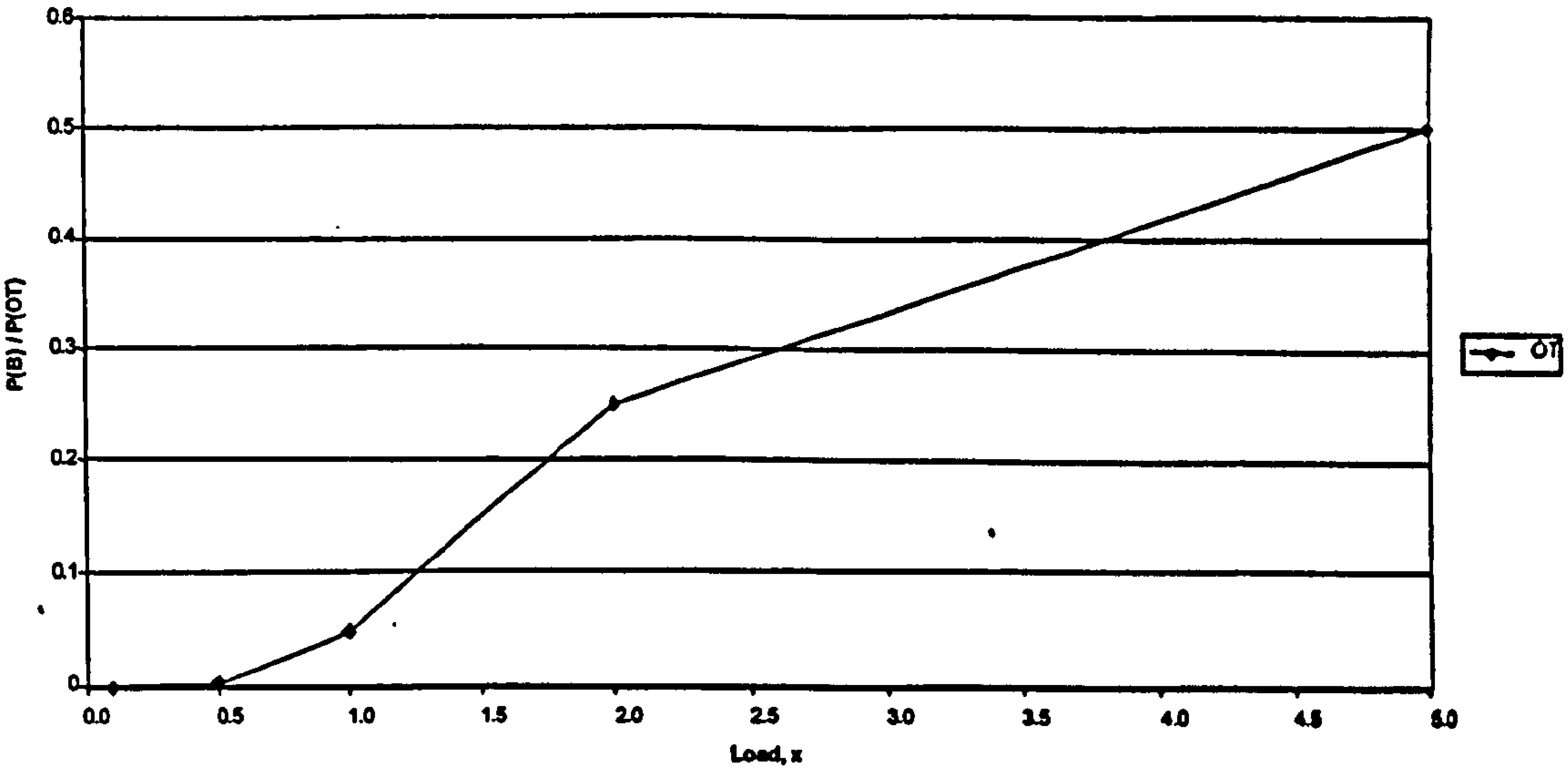


Figure E.20 Fragility curves for high ground (non-raised embankments)

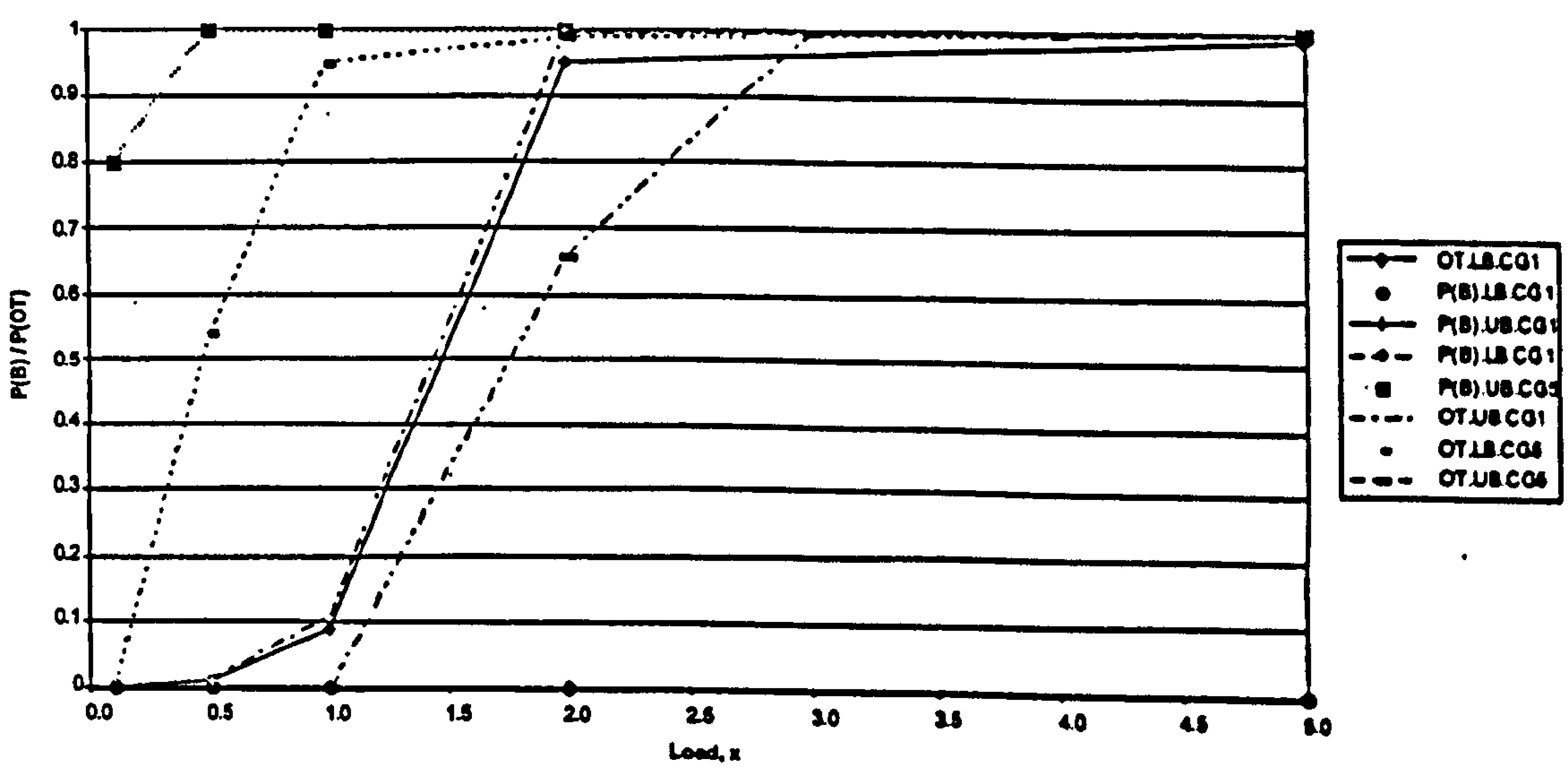


Figure E.21 Fragility curves for culverts

E.4. TYPE 5: VERTICAL SEAWALLS

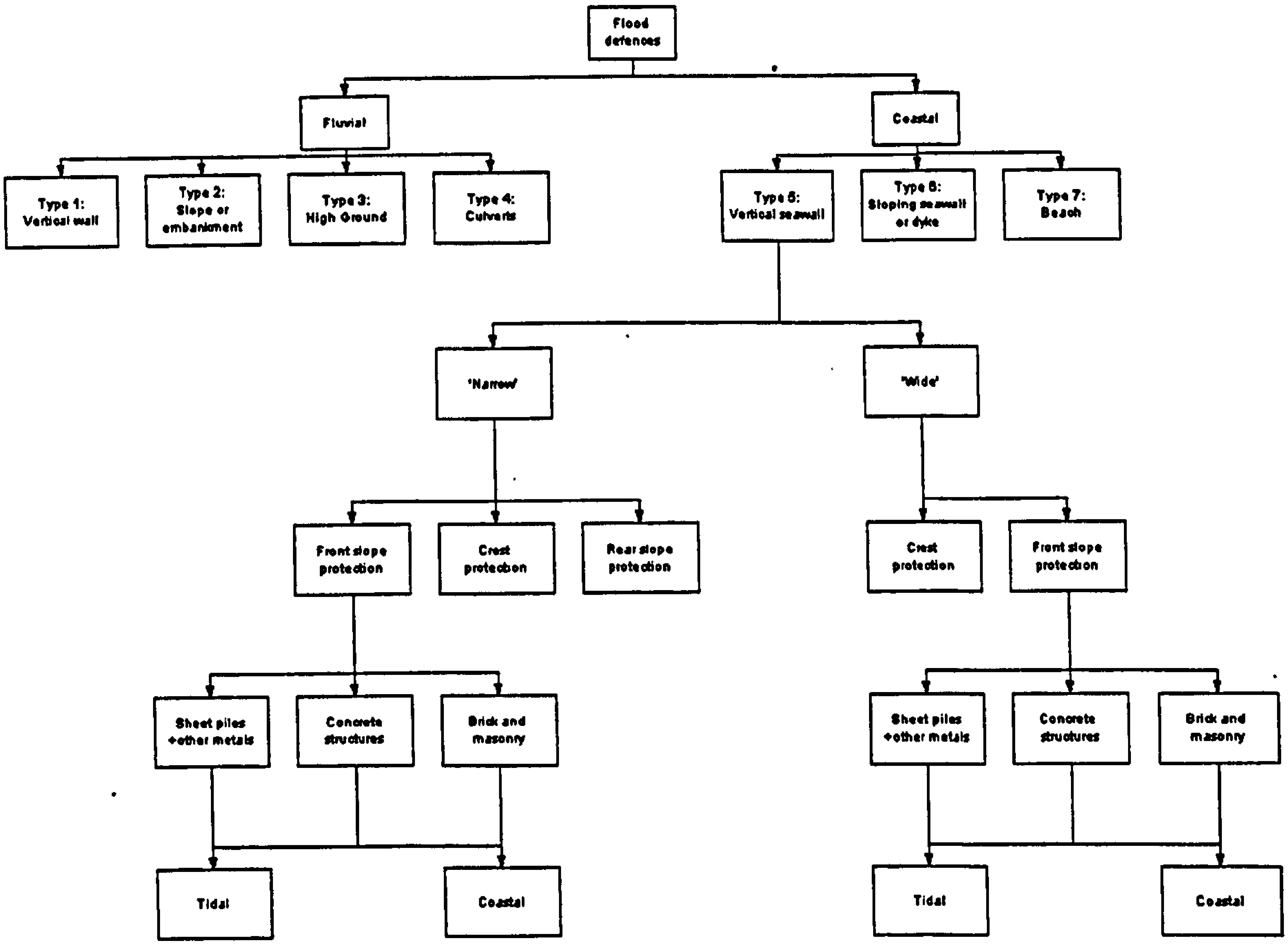


Figure E.22 Detailed classification of vertical coastal defences

Table E.4 Classification description and associated NFCDD codes for vertical coastal defences

Classification	Relevant NFCDD codes			
	Type	Sub-type	Material	Revetment
Vertical sea wall				
Sheet piles and	Seabed/ Foreshore: CB/FS		I/J	-
other metals	Defence: CS/FI/FC/FO/(DO)	W	P/L/S	-
Concrete	Seabed/ Foreshore: CB/FS		I/J	-
structures	Defence: CS/FI/FC/FO/(DO)	W	C/D/Q	-
Brick and	Seabed/ Foreshore: CB/FS		I/J	-
masonry	Defence: CS/FI/FC/FO/(DO)	W	M	-

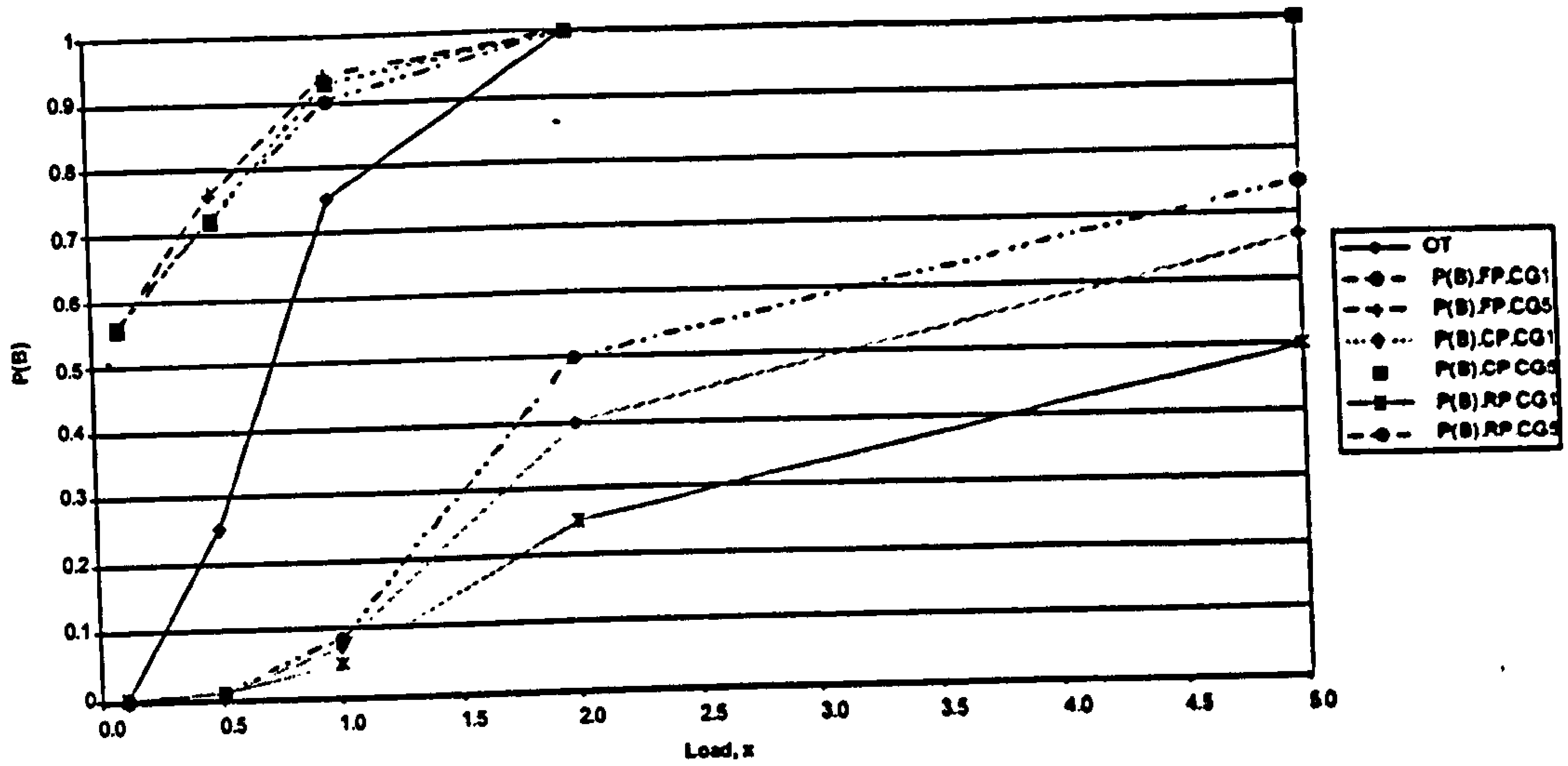


Figure E.23 Fragility curve for vertical coastal defence: narrow, front protected by sheet piles

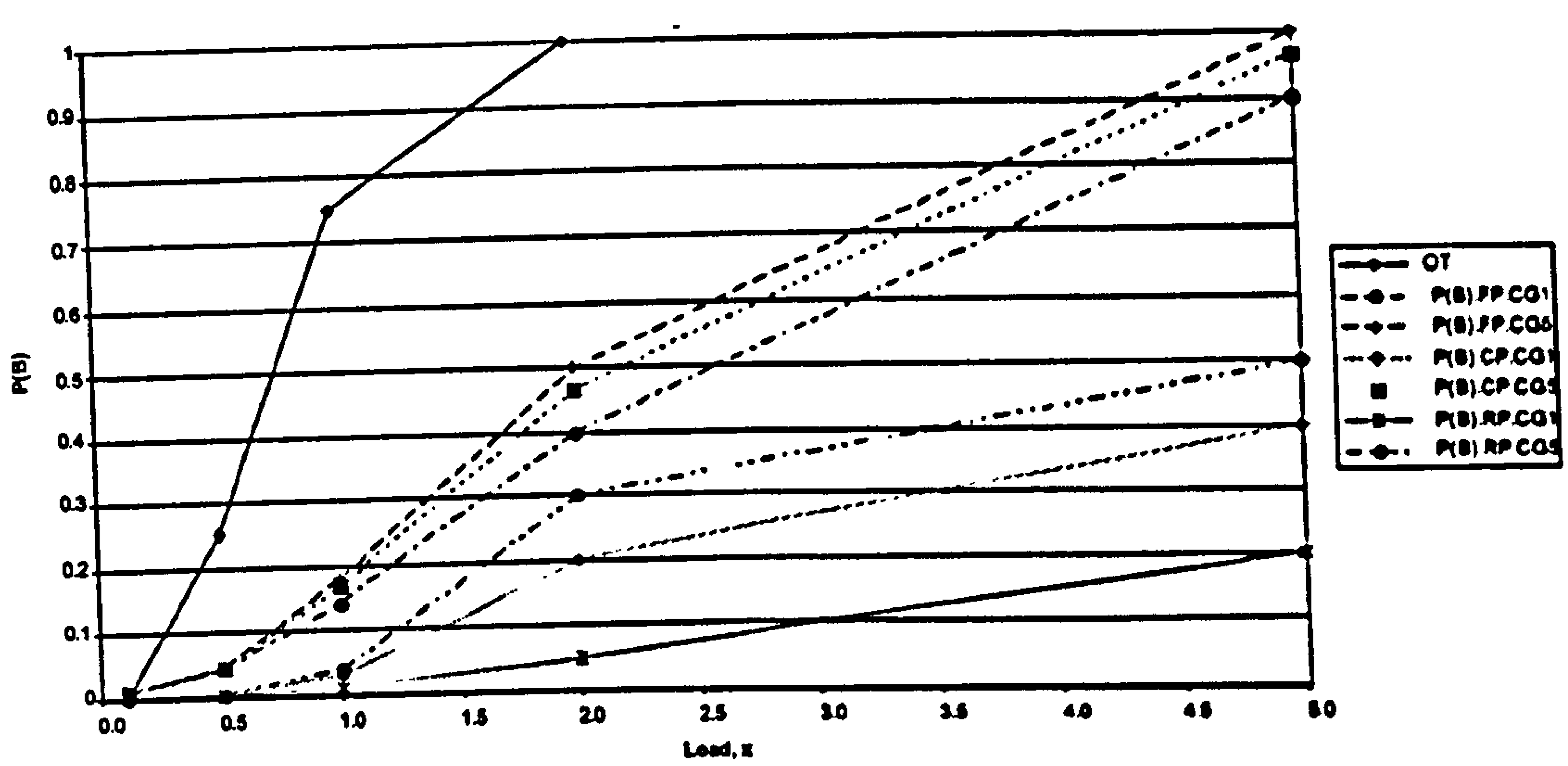


Figure E.24 Fragility curve for vertical coastal defence: narrow, front protected by concrete

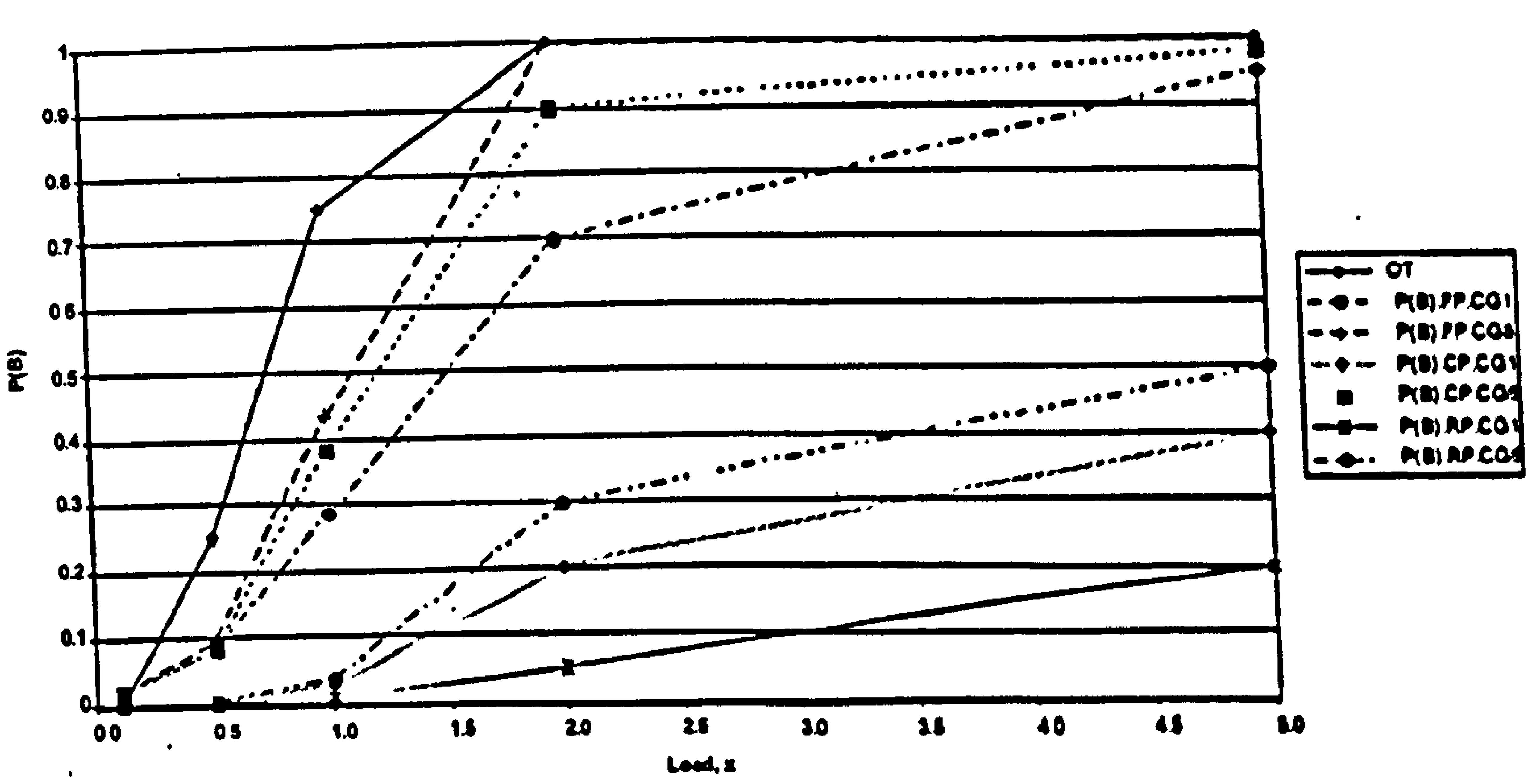


Figure E.25 Fragility curve for vertical coastal defence: narrow, bricks & masonry front

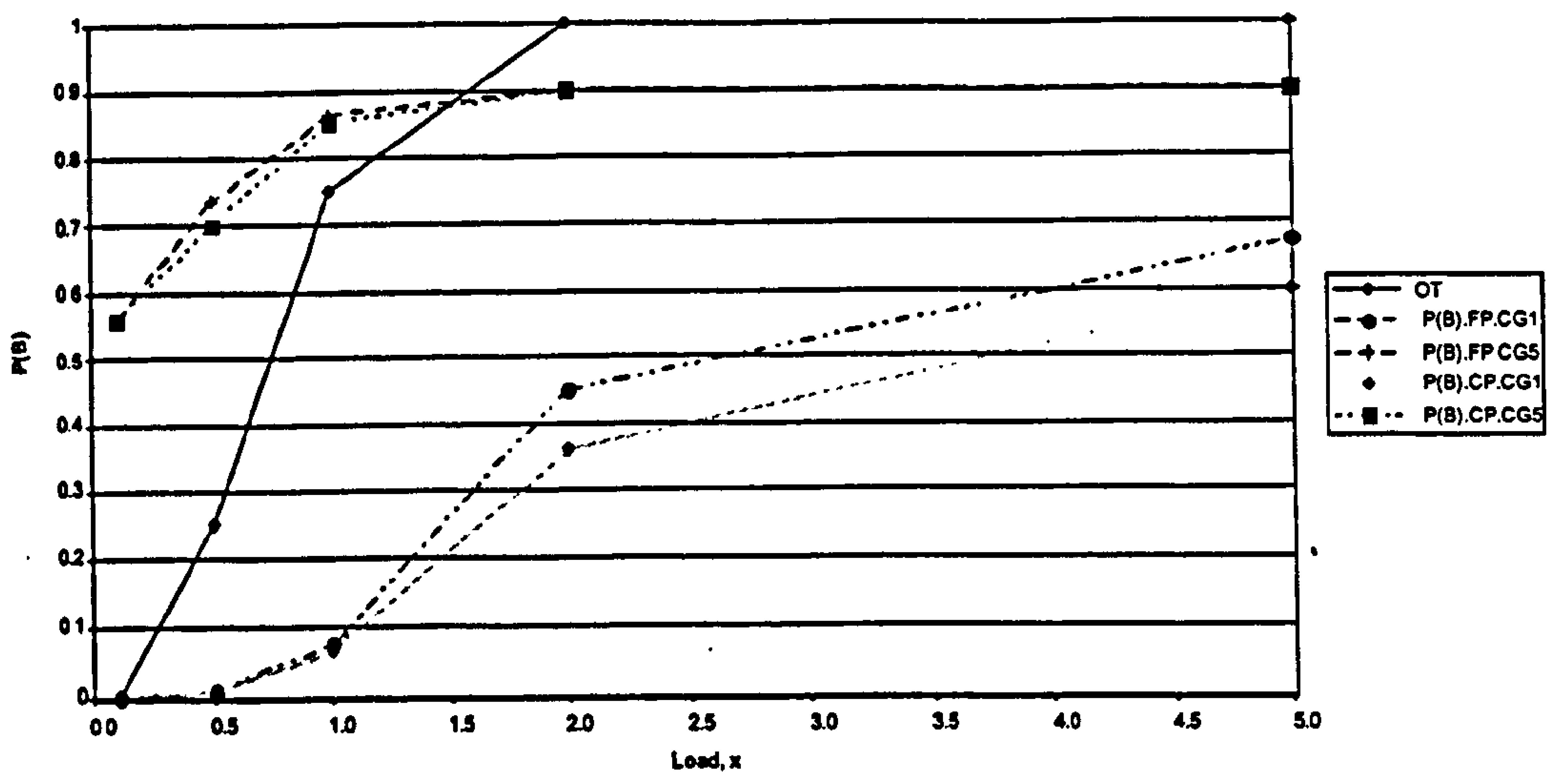


Figure E.26 Fragility curve for vertical coastal defence: wide, front protected by sheet piles

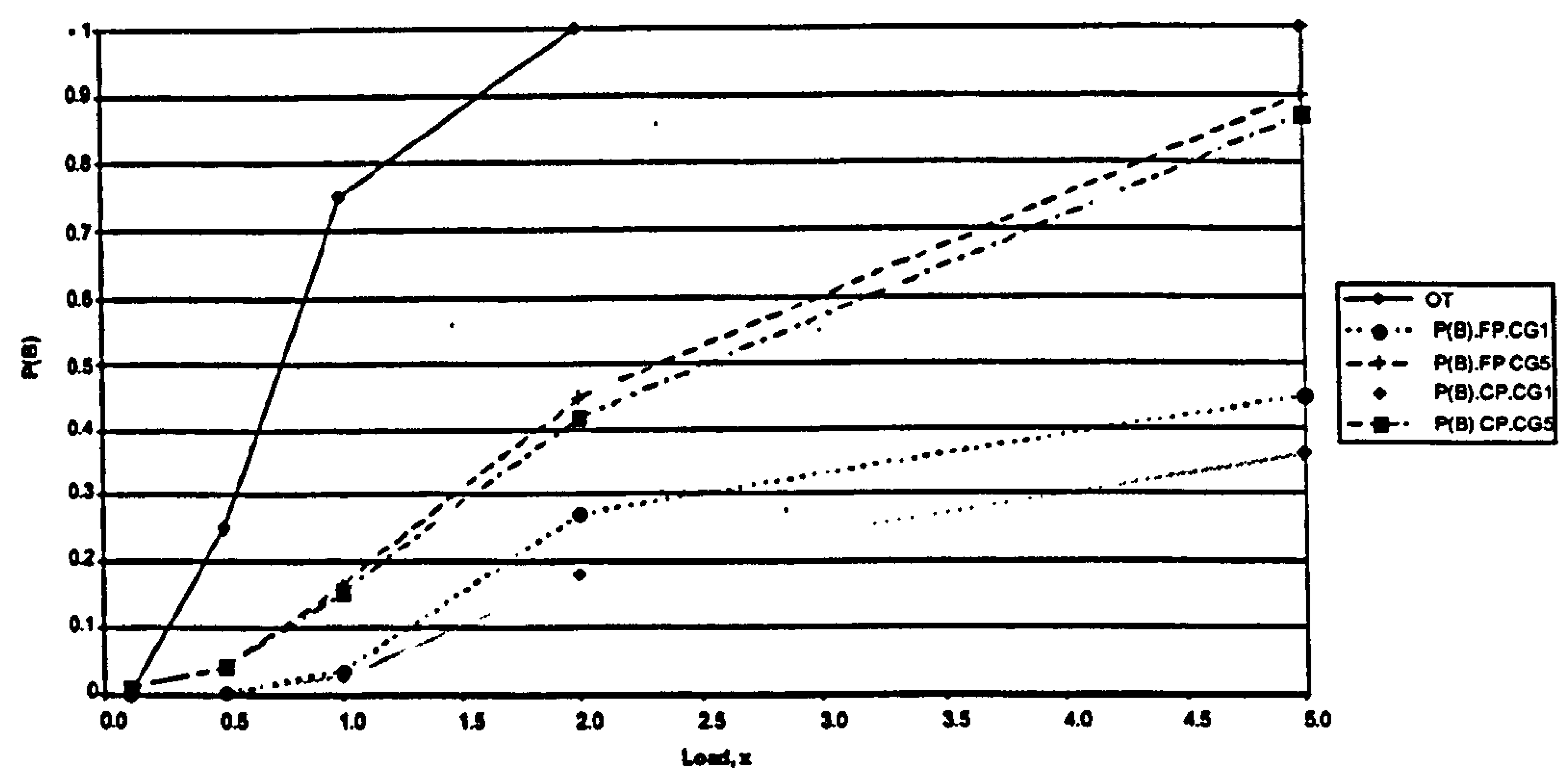


Figure E.27 Fragility curve for vertical coastal defence: wide, front protected by concrete

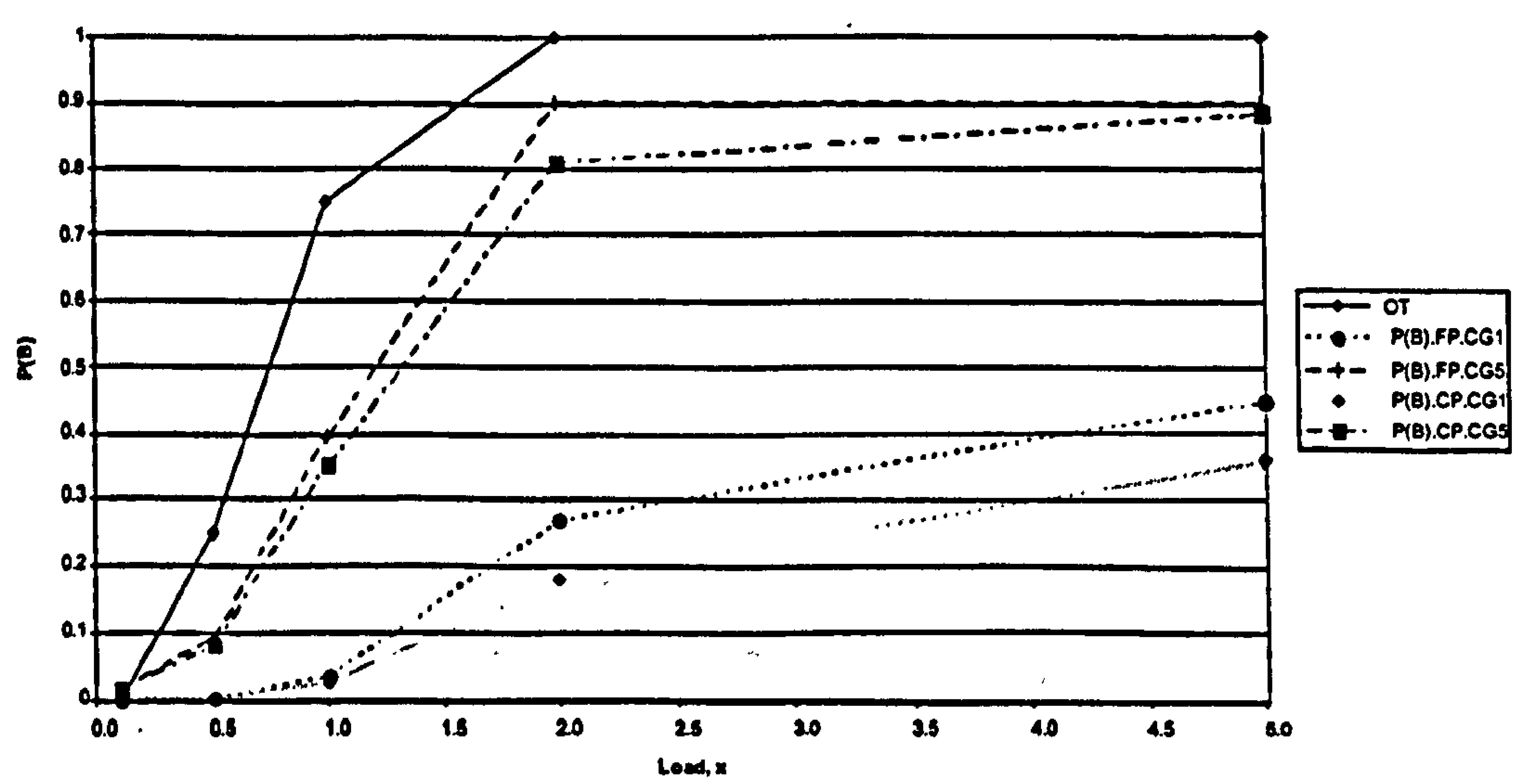


Figure E.28 Fragility curve for vertical coastal defence: wide, bricks and masonry front

E.5. TYPE 6: SLOPING SEAWALLS OR DYKES

Note: Permeable revetments include rock armour, impermeable revetments include asphalt.

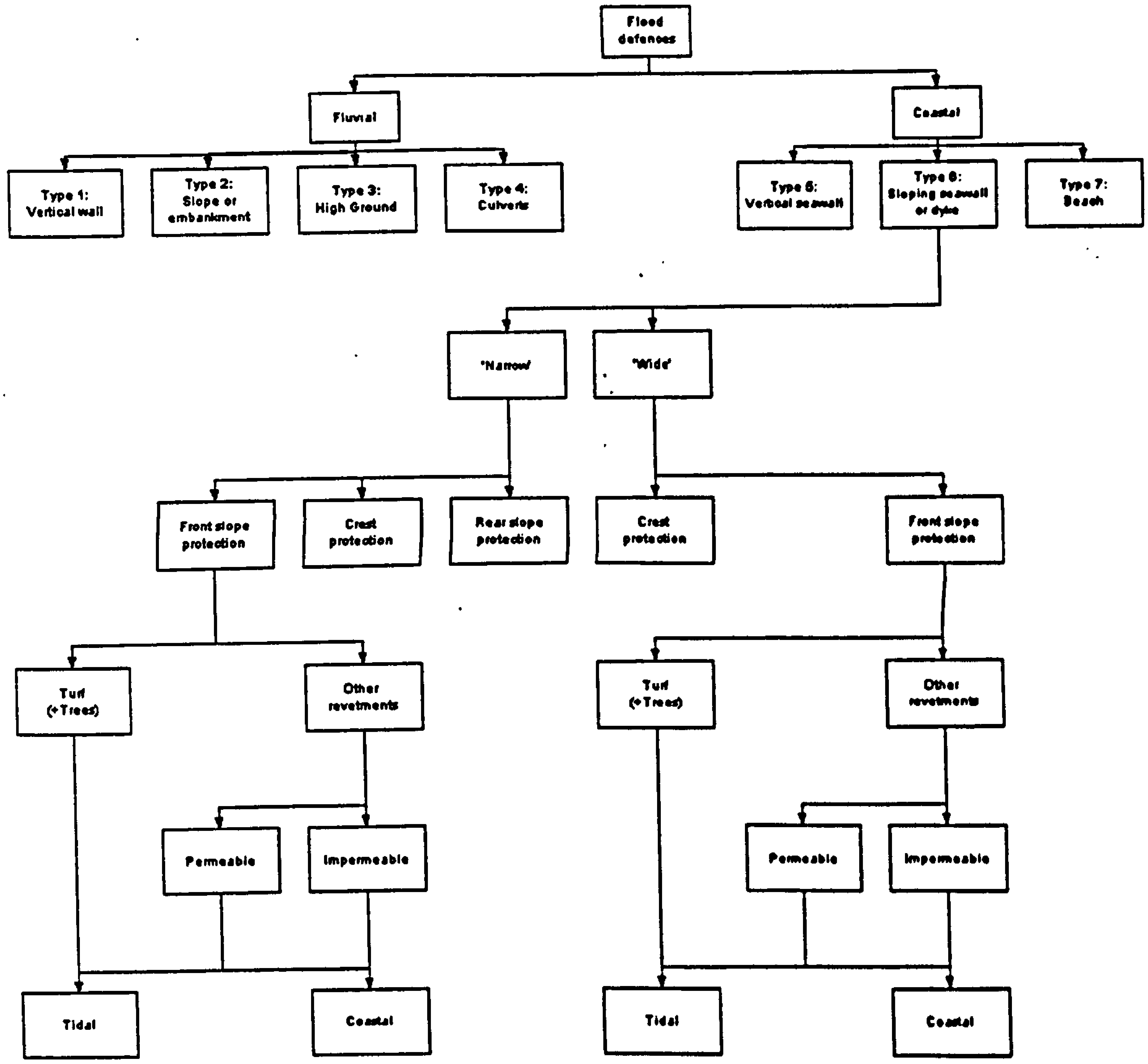


Figure E.29 Detailed classification of sloping coastal defences

Table E.5 Classification description and associated NFCDD codes for sloping coastal defences

Classification	Type	Sub-type	Material	Revetment
Sloping seawall or dyke	Seabed/ Foreshore: CB/FS		I/J	-
	Defence: CS/FI/FC/FO/(DO)	B	A/B/C/D/E/L	Permeable: F/I1/J/K/T Impermeable: U/W/Y Either: A/B/O/Z

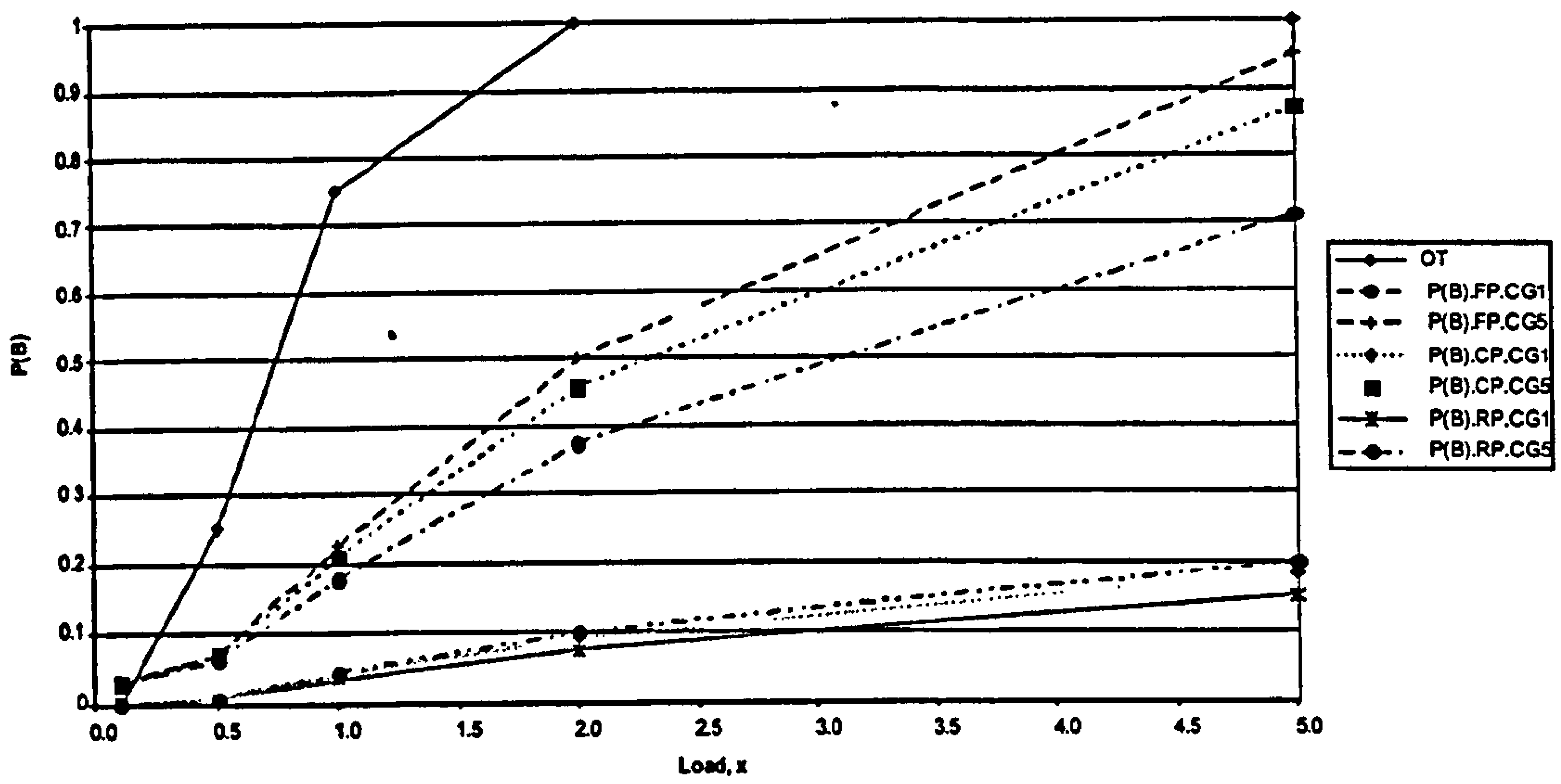


Figure E.30 Fragility curve for sloping coastal defence: narrow, front protection permeable

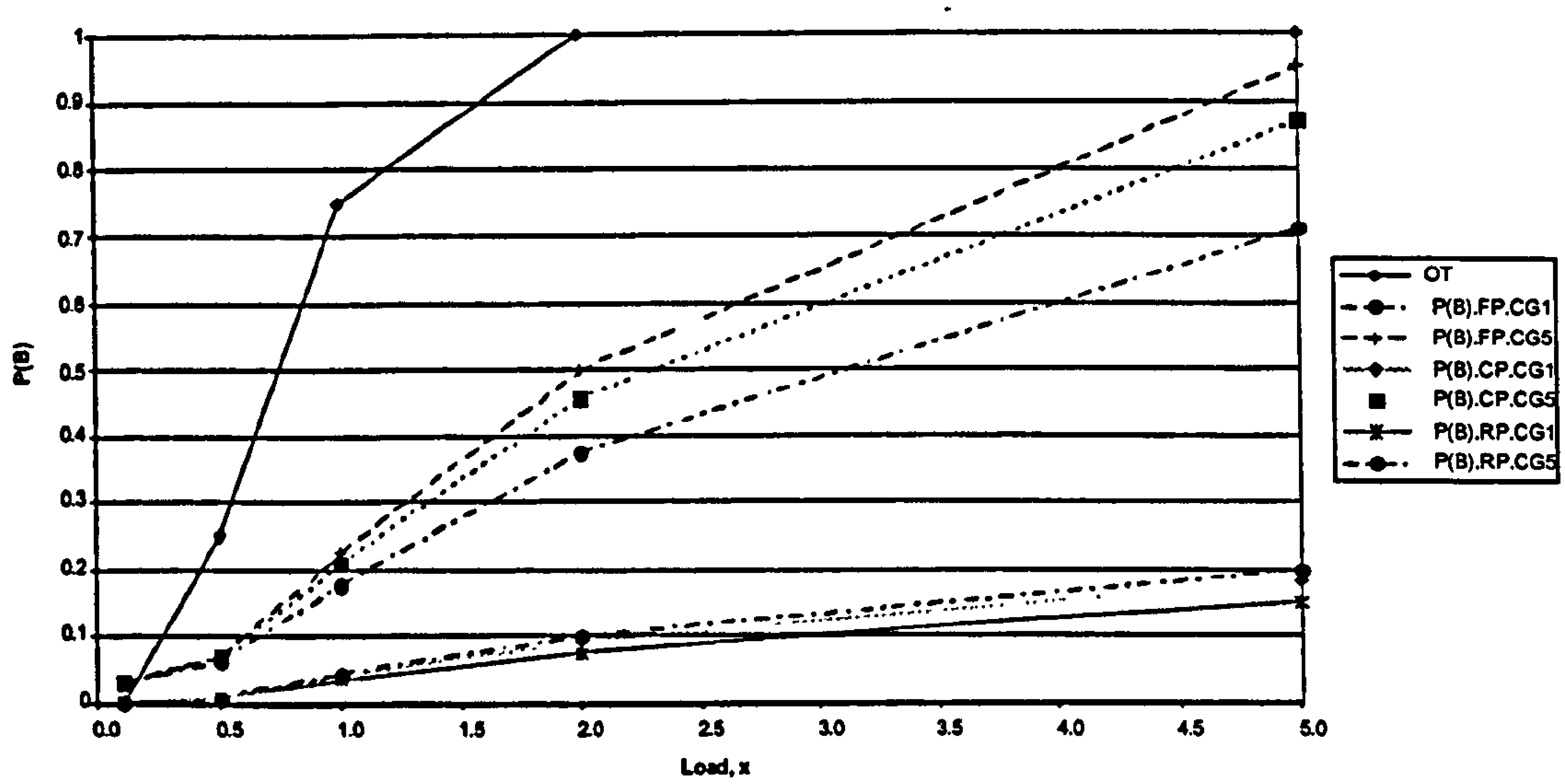


Figure E.31 Fragility curve for vertical coastal defence: narrow, front protection impermeable

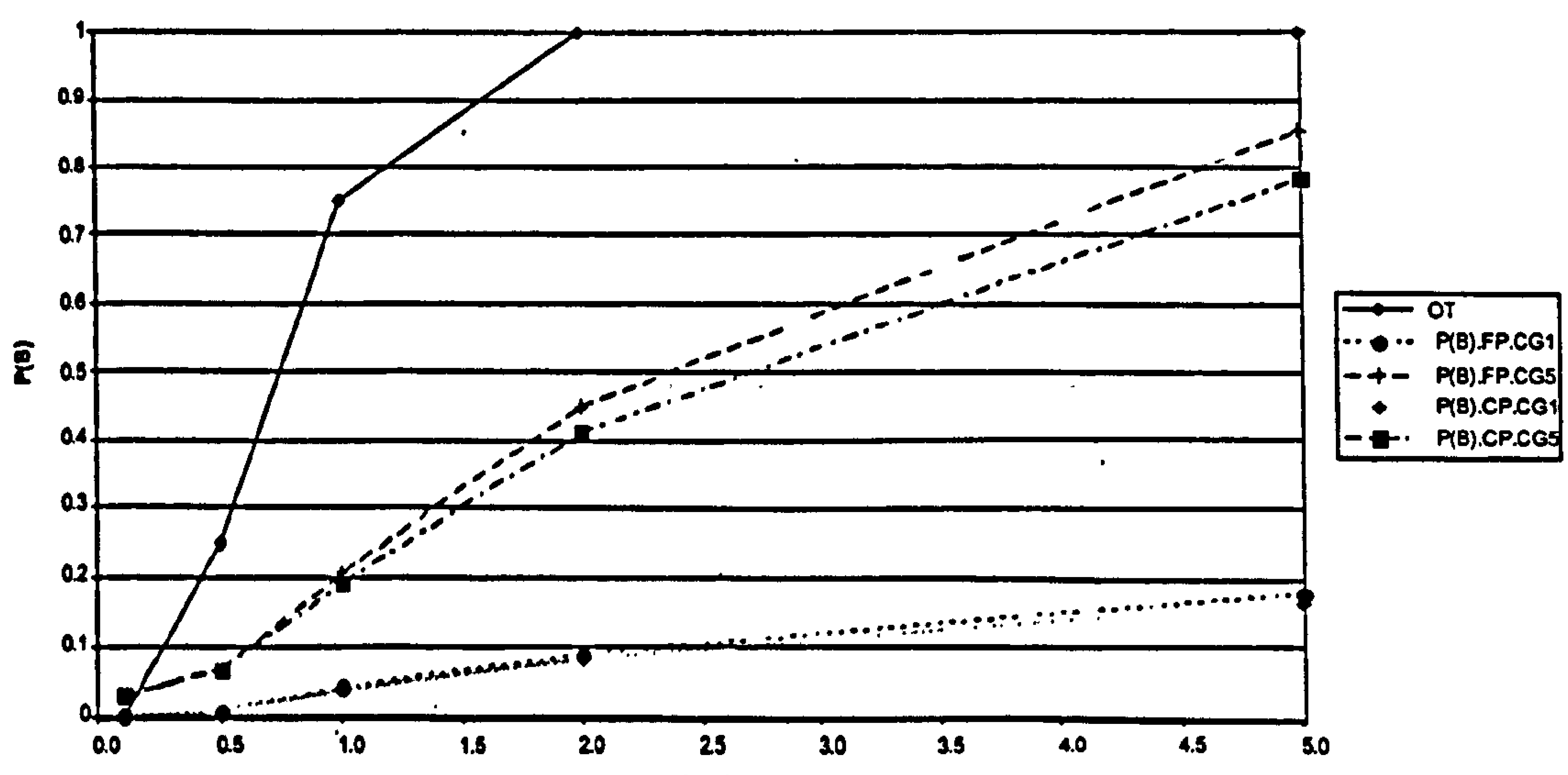


Figure E.32 Fragility curve for vertical coastal defence: wide, front protection permeable

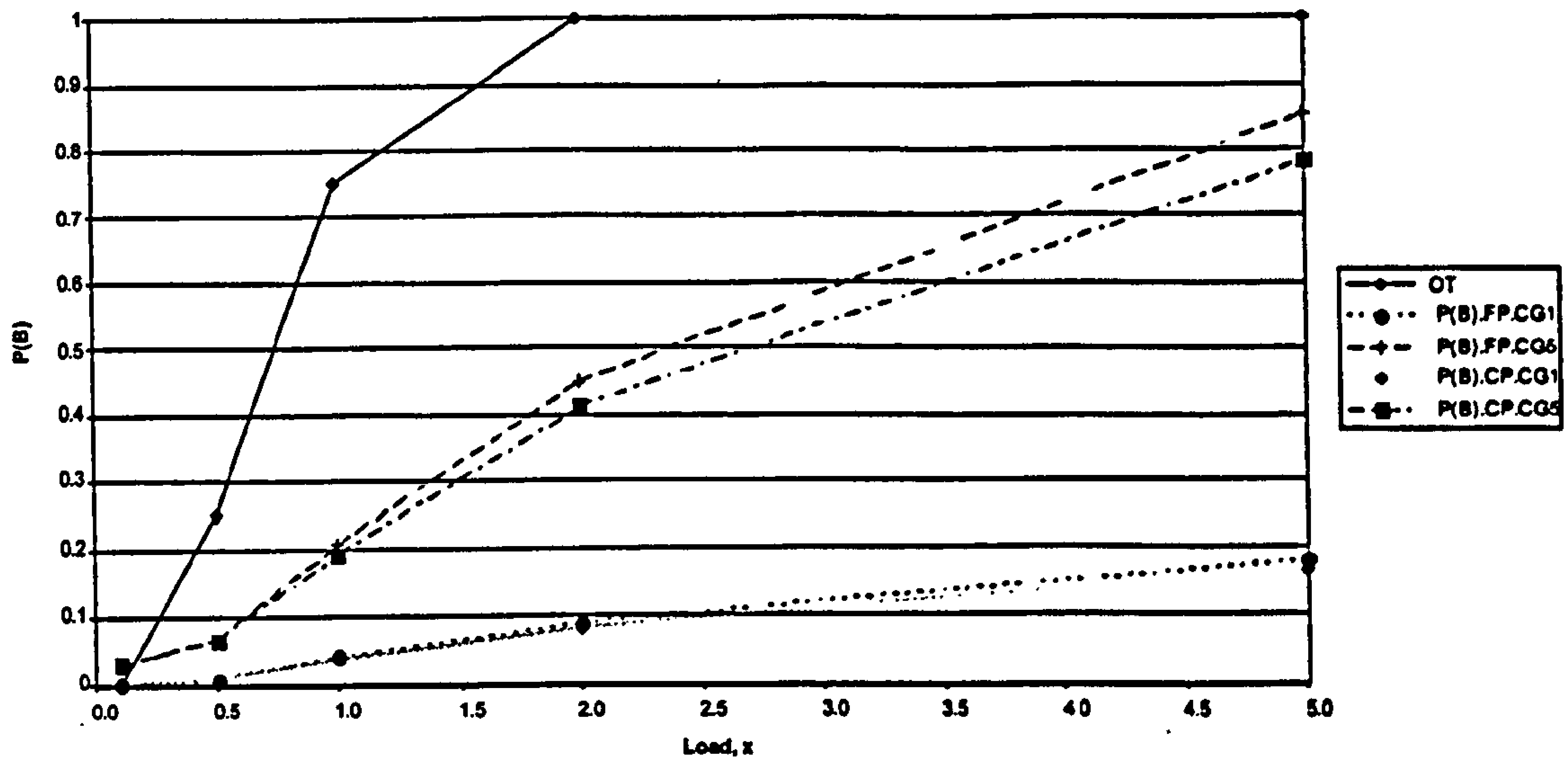


Figure E.33 Fragility curve for vertical coastal defence: narrow, front protection impermeable

E.6. TYPE 7: BEACHES

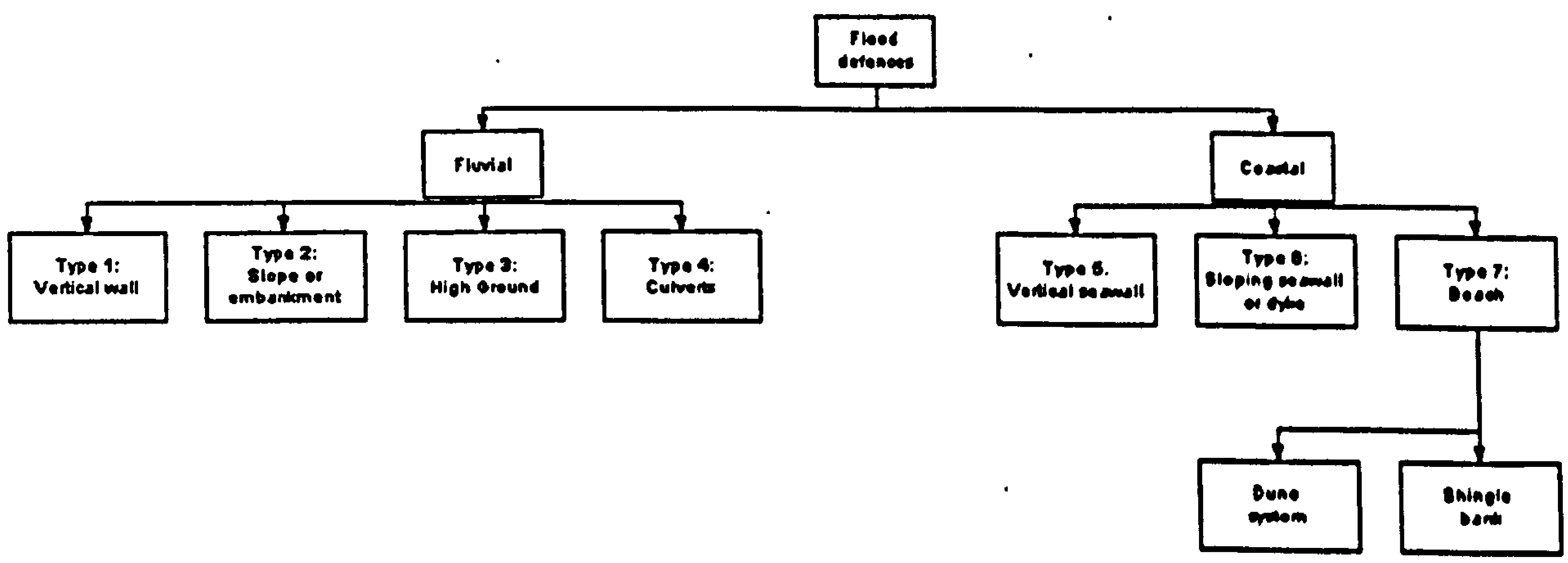


Figure E.34 Detailed classification of beaches

Table E.6 Classification description and associated NFCDD codes for beaches

Beach	Type	Sub-type	Material	Revetment
Sand / dune system	FS/DU	B	I	-
Shingle bank	FS/FI/FC/FO	B	J	-

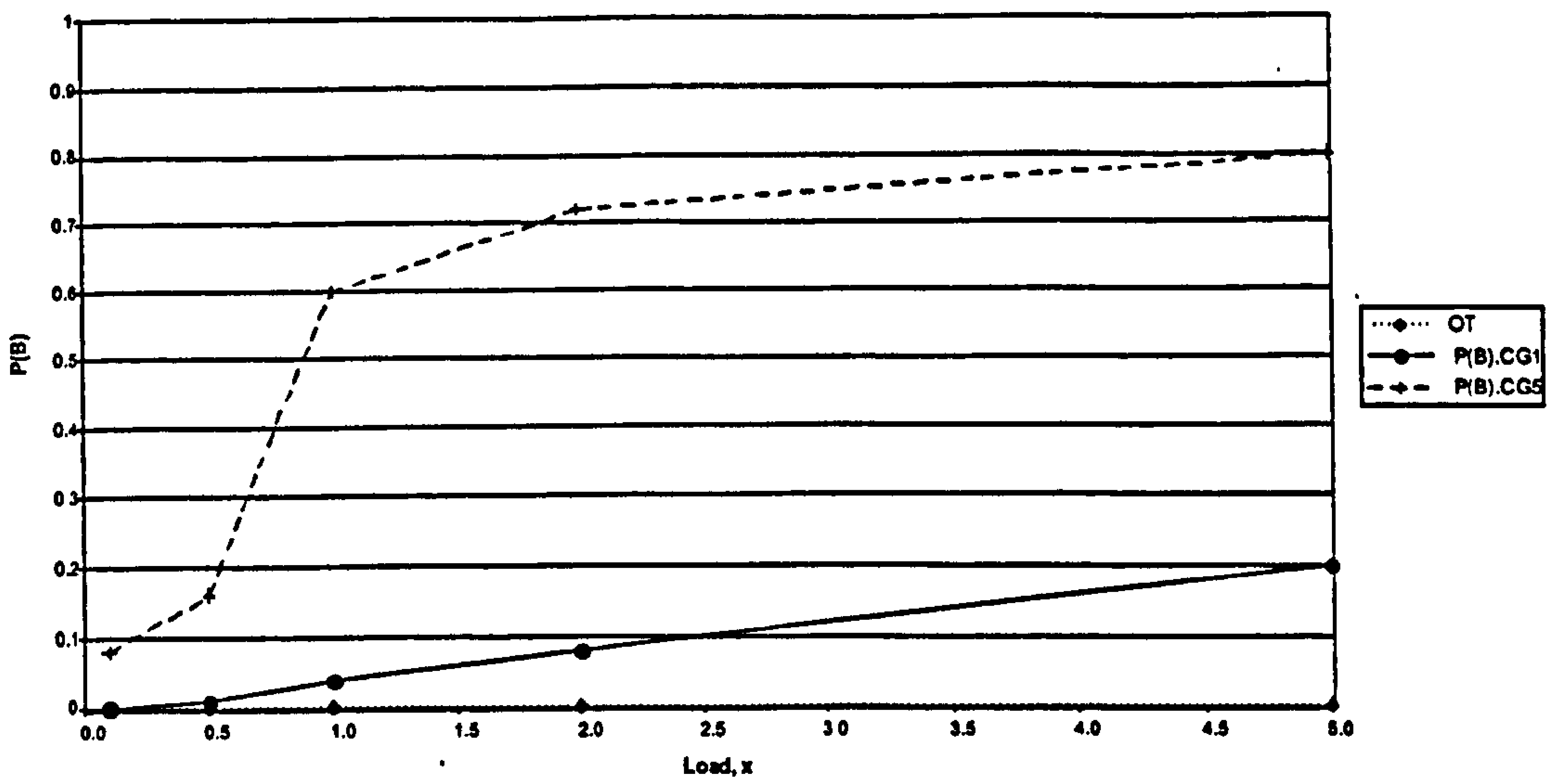


Figure E.35 Fragility curve for dunes

E.7. CROSS-SECTION DEFENCES AND THEIR INFLUENCE ON FRAGILITY

River defences frequently have outfalls, flap valves, penstocks and sluice gates placed within them. These structures are often the point of failure for many such defences and as a result of this need to be taken into account when calculating defence fragility and therefore considered in the classification.

This weakening of the structure can be accounted for by a change in the structure's fragility. Defences with cross-sectional structures can be identified by querying the NFCDD. The NFCDD codes associated with these types of structures are shown in Table E.7. Whilst this was recognised as being important no reduction factor was included in the high level methodology and it was identified as an area for further research.

Table E.7 The NFCDD codes associated with outfalls, flap valves, penstocks and sluice gates

Outfalls	Type	Sub-type	Material	Revetment
Flap Valves	OI/OO/OM/OP	F	B/L/S	-
Penstocks	OI/OO/OM/OP	P	B/L/S	-
Gates	OI/OO/OM/OP	O/G	B/L/S	-
Screens	OI/OO/OM/OP	K	L/S	-

E.8. NFCDD CODES

The following are the asset identification codes that are used to classify the defences into generic types as described above. Codes in grey are not flood defence structural elements.

Name: **ASSET ELEMENT TYPE CODE**
Definition: **This code uniquely identifies the asset element type.**
Values: AP-Abstraction Point
AR-Apron
BA-Bastion

BE-Berm
FM-Farm Access Bridge
FB-Foot Bridge
RA-Rail Bridge
RB-Road Bridge
BR-Breakwater
CA-Canal/ River Aqueduct
CB-Channel Bed
CS-Channel Side
FC-Crest
CU-Culvert
DC-Debris Collector
DD-Drainage Ditch
DO-Delph Ditch or Landward Slide
DU-Dune
FI-Inward Face
FO-Outward Face
FD-Ford
FS-Foreshore
FG-Flood Gate
GB-Gauging Board
GS-Gauging Station
GR-Groyne
HG-High Ground
LK-Lock
MH-Manhole
OI-Outfall Inward Face
OO-Outfall Outward Face
OM-Outfall Mechanism
OP-Outfall Protection
PI-Pipe or Water Main
PS-Pumping Station
DS-Debris Screen
WS- Weed Screen
SW-Seawall
SC-Service Crossing
AT-Road Tunnel
RT-Rail Tunnel
WE-Weir
LI-Lifting Weir

Name: **ASSET ELEMENT SUB TYPE CODE**
Definition: **Code used to define an asset element sub-type.**
Values: (Blank)-None
AP-Apron
CA-Aqueduct (Canal)
X-Automatic
BR-Breastwork
AB-Bridge (Arched)
FD-Bridge (Flat Deck)
FM-Crossing (Farm Bridge)
FB-Crossing (Foot Bridge)
RA-Crossing (Rail Bridge)
RB-Crossing (Road Bridge)
BC-Culvert (Box)
PI-Culvert (Pipe)
D-Dropboard
B-Embankment

E-Electricity
HU-(Dune) Fence/Hurdle
F-Flap Valve
G-Gas
O-Opening Gate
S-Sluice Gate
GE-Geotextile
H-Headwall
L-Lined
M-Manual
N-Natural
Y-Oil
U-Oval
P-Penstock
PG-Piling
A-Rectangular
R-Regraded
C-Round
K-Screen
SW-Splash Wall
Q-Square
T-Telecom
TP-Tetropod*
V-Valve Gate
VE-Vegetated
W-Wall
CO-Complex
Z-Other

*Tetropod is used to represent all man made armour units

Name: **ASSET ELEMENT MATERIAL TYPE**
Definition: **Material from which asset is constructed.**
Values: (Blank)-None
A-Asbestos Cement
B-Plastic
C-Concrete - Cast Insitu
D-Concrete - Precast
E-Earth
F-Corrugated Steel
G-Gabion or Reno Mattress
H-Timber Pile
I-Sand
J-Shingle
K-Spiling
L-Aluminium
M-Masonry or Brick
N-Faggoting
O-Other (Describe)
P-Steel Piling
Q-Concrete Piling
R-Rock or Stone
S-Steel/Iron
V-Clay
W-Timber
X-Complex (Describe)
Z-Bagwork

Name: **ASSET ELEMENT REVETMENT CODE**
Definition: **Code describing the facing material protecting an asset element.**
Values:
(Blank)-None
A-Asphalt
B-Bituminised Aggregate
C-Concrete Cladding
J-Concrete Granulated
O-Precast Concrete
Y-Concrete (Caps)
D-Fabric
G-Gabion Toe
P-Piles
S-Plastic
R-Reno Mattress
H-Grouted Stone
U-Unworked Stone or Rip-rap
W-Worked Stone or Concrete Slab Cladding
K-Trees
T-Turf or Grass
E-Wood
F-Foundary Slag
Y-Concrete Capping
Z-Other

References

ENVIRONMENT AGENCY (1996), Risk Assessment for Sea and Tidal Defence Schemes. Report 459/9/Y, Bristol.

ENVIRNONMENT AGENCY (2001), National Flood and Coastal Defence Database: Data catalogue, Issue 1.0, Environment Agency, Bristol.

Appendix F

High Level inundation modelling*

Having estimated the probability of every scenario of defence failure, the consequences of flooding are established by first estimating floodplain depths and finally, the damages resulting from a flood of this depth. The key steps are discussed below. A summary of the equations is then provided in Table F.1. An example of each parameter is then given in Table F.2 and
Table F.3.

Overflow flood volumes – fluvial flood plains

In the case of overflow of a defence, the flow of water over or through the defence is similar to the flow over a rectangular, broad-crested weir and, therefore, the peak discharge Q_p is given by (French, 1994):

$$Q_p = 1.71 b h_{\max}^{1.5} \quad \text{(F.1)}$$

where b is flow width and h_{max} is the maximum head over the defence crest or breach, which is an empirical function of the form :

$$h_{\max} = B_1 x^{C_1} \quad \text{(F.2)}$$

where B_1 and C_1 are parameters based on type of failure (HR Wallingford *et al.*, 2002). As previously, x is the ratio of the return period of the event under consideration over the SOP of the defence, and can take any value between 0 and 3. An upper limit to the ratio x has been defined to reflect the natural limit to h_{max} over any single defence. Above this limit overtopping of upstream defences is likely to limit the continued increase in h_{max} with increasing load. The flood volume from failure of defence section i, V_i, is estimated as:

$$V_i = 0.5 Q_p T_r(x) \quad \text{(F.3)}$$

where $T_r(x)$ is duration of flow across the defence, defined by the function:

$$T_r(x) = A^{0.25} B_2 x^{C_2} \quad \text{(F.4)}$$

where A is the catchment area and B_2 and C_2 are empirical parameters depending on floodplain type and return period of the event (HR Wallingford *et al.*, 2002).

* The reader is reminded that the flood risk assessment methodologies were created in conjunction with external collaborators. The parametric inundation routine used in the high level methodology was provided by HR Wallingford Ltd. and is not the work of the author. It is included in this thesis for completeness. The reader is referred to HR Wallingford *et al.* (2002) for a more detailed study of the methodology.

As shown above both the duration of the flow across the defence (*i.e.* T_r) and depth of flow over defence (*i.e.* h) depend on the ratio, x. Therefore, both duration and water depth will increase as the ratio x increases.

Overtopping flood volumes – coastal flood plains

The coastal overtopping volume, V_s, is a function of both waves and water levels and so the volume depends on the overtopping rate q per unit length of defence:

$$V_i = qbT_s(x) \tag{F.5}$$

where b is the width of the defence overtopped, $T_s(x)$ is the effective duration of overtopping in seconds and the overtopping rate per unit length of defence is approximated from:

$$q = B_3 x^{C_3} \tag{F.6}$$

The rate q=0.05m^3/m/s at x=1 is taken as the point at which significant overtopping occurs in an event equal to the SOP and the value of C_3 is extracted by fitting to the results of typical overtopping analyses (CIRIA and CUR, 1991). The duration of coastal storms is strongly influenced by the rise and fall of the tide and can be approximated (in hours) as:

$$T_s(x)=x.T_s \tag{F.7}$$

Where T_s=3 hours, which is the typical effective duration of an overtopping event during a storm equal to the SOP. At the upper limit, x=3, the duration of the overtopping event will be 9 hours. In practice this depends on the relative magnitude of the tide and surge residual (Pugh, 1987), but in the absence of strong evidence to the contrary this has been found to be a good first approximation.

Breach flood volumes – fluvial and coastal floodplains

Overtopping is assumed to occur over the entire defence length S_d, whereas breach width, b_B, is assumed to be a function of the load x:

$$b_B=x.C_b.S_d\,,\ b_B \le S_d \tag{F.8}$$

where C_b is an empirical constant (HR Wallingford *et al.*, 2002). There is little information on the breaching process. Therefore the estimation of the constant C_b has been supported by empirical evidence. For example, during the Autumn 2000 floods in England and Wales a river wall in Lewes with a SOP of 30-50 years breached along 33m during a 70-100 year return period flood event. The flood volume is then estimated from Equations (F.3) or (F.5), using $T_r(x)$ values from Equation (F.4), and $T_s(x)$ values from (F.7) for coastal defences.

Table F.1 Summary of equations for calculating representative breach widths, maximum head and duration of flow across a defence (HR Wallingford et al., 2002)

Failure mode	Defence Type	b	h or q	T
Overtopping	Fluvial	$b_{OT} = L_{def}$	$h = x^2 B_1$	$Tr = x^{0.5} A^{0.25} B_2$
	Coastal	$b_{OT} = L_{def}$	$q = x^{1.5} B_3$	$Ts = x\, B_4$
Breaching	Fluvial	$b_B = x\, B_5 L_{def}$	$h = x^{0.5} B_1$	$T = x^{0.5} A^{0.25}\, B_2$
	Coastal	$b_B = x\, B_5 L_{def}$	$h = x^{0.5} B_1$	$T = x\;\; B_5$

where:

L_{def}	=	Defence length (m)
x	=	Ratio between the return period of the event for which the defence is overtopped / breached and SOP of the defence (limited to a maximum value of 3)
$T(x)$	=	Duration of flow across the defence(s) (hours)
H	=	Maximum head on breach or defence crest (m)
Q	=	Flow per unit length of defence ((m^3/sec)/m)
B	=	Representative breach width or length of defence overtopped (m)
A	=	Catchment area (km^2).
B_5	=	Base value of b, determined when the return period is equal to the *SoP*, assumed to be 0.05 for fluvial coastal defences
B_1	=	Base value of h, determined when the return period is equal to the *SoP*, assumed to be 0.05m for overflow / overtopping scenario, and 0.5m for breach scenario
B_3	=	Base value of q, determined when the Load is equal to the *SoP*, assumed to be 0.05 (m^3/sec)/m
B_2	=	constant, which has the following values 1.2 hours/km$^{0.5}$ for steep floodplains and Return Period of the event > 50 years 0.6 hours/km$^{0.5}$ for steep floodplains and Return Period of the event =< 50 years 1.2 hours/km$^{0.5}$ for shallow floodplains and Return Period of the event > 50 years 0.6 hours/km$^{0.5}$ for shallow floodplains and Return Period of the event =< 50 years 0.8 hours/km$^{0.5}$ for average floodplains and Return Period of the event > 50 years 0.4 hours/km$^{0.5}$ for average floodplains and Return Period of the event =< 50 years
B_4		3.0 hours for coastal floodplain

Table F.2 Estimation of b, h and T parameters for a defence that has been overtopped (HR Wallingford et al., 2002)

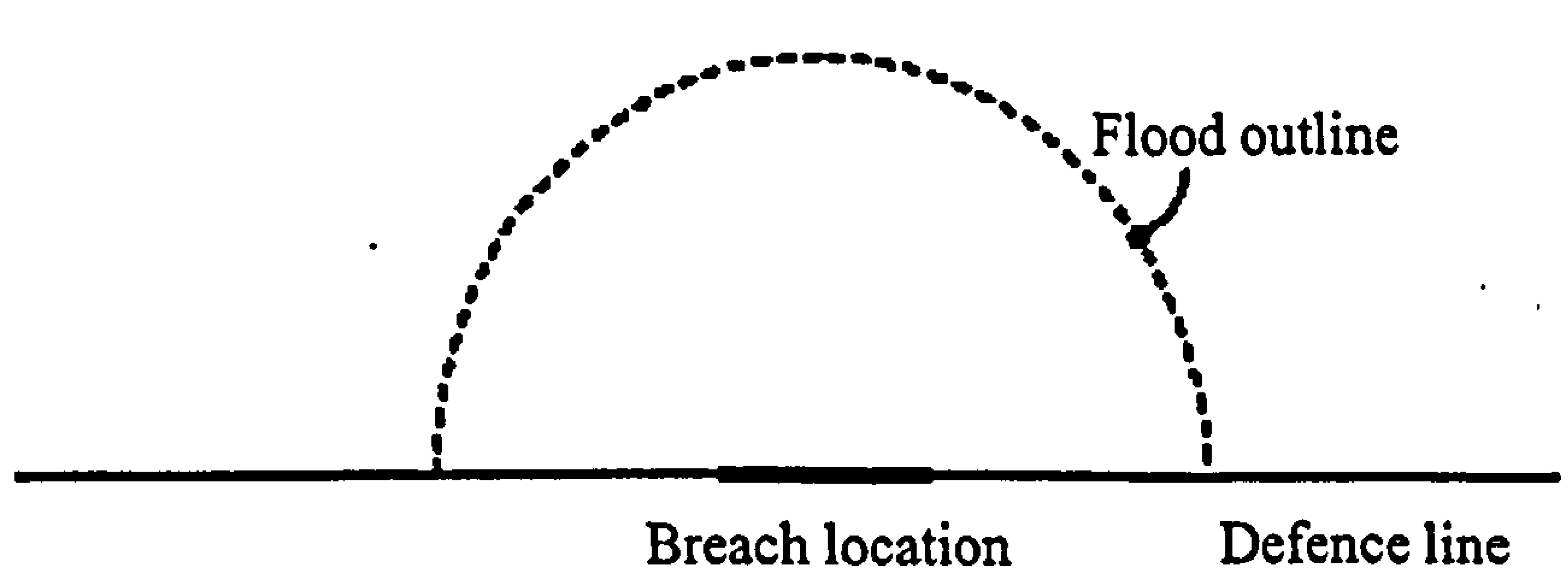
Valley type	**L_{def} (m)**	**SOP (yrs)**	**RP event (yrs)**	**Load x**	**b (=L def) (m)**	**h (m)**	**q (m3/sec/m)**	**T (hrs)**
Fluvial "Average slope"	306	100	1000	3	306	0.15		3.78
			500	3	306	0.15		3.78
			200	2	306	0.10		3.09
			100	1	306	0.05		2.18
			50	0.5	306	0.03		0.77
Coastal	400	100	1000	10	400		1.58	30.00
			500	5	400		0.56	15.00
			200	2	400		0.14	6.00
			100	1	400		0.05	3.00
			50	0.5	400		0.02	1.50

Table F.3 Estimation of b, h and T parameters for a defence that has been breached (HR Wallingford et al., 2002)

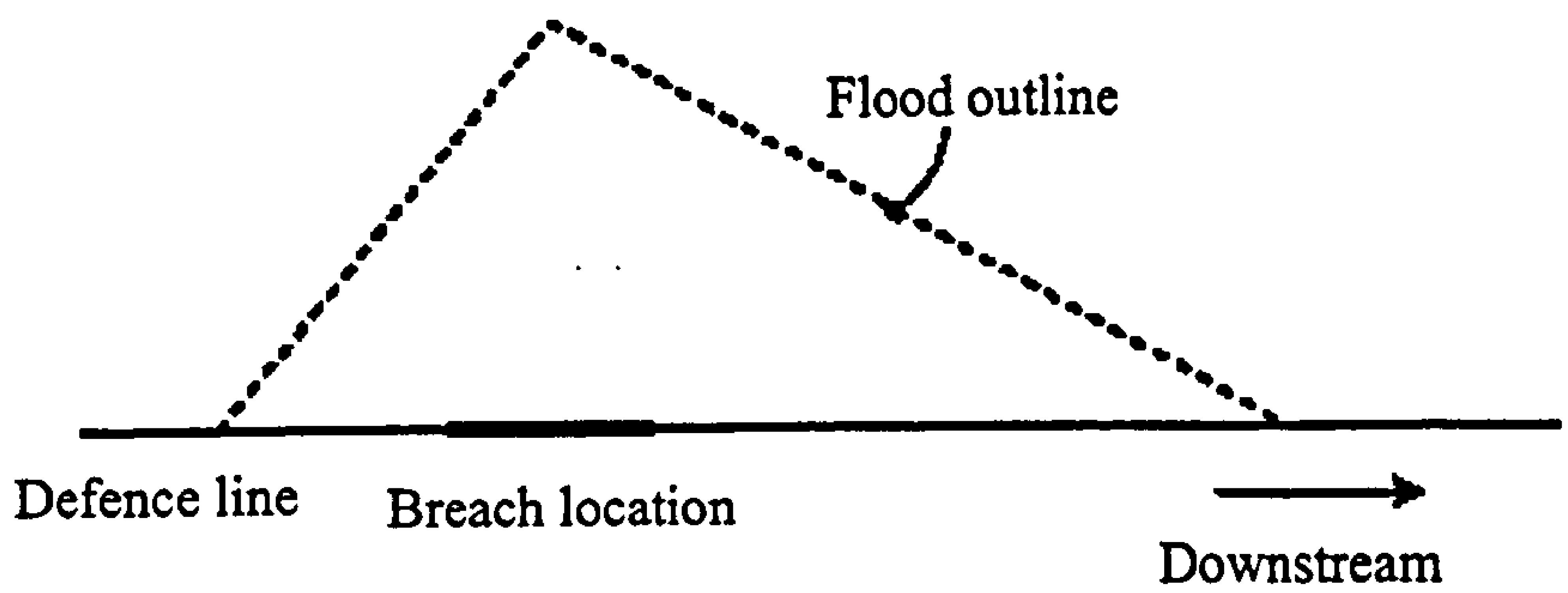
Valley Type	L_{def} (m)	SOP (yrs)	RP event (yrs)	Load x	b (m)	h (m)	T (hrs)
Fluvial	36	50	1000	3	5.4	0.9	3.78
			200	3	5.4	0.9	3.78
"Average slope"			100	2	3.6	0.7	3.09
			50	1	1.8	0.5	1.09
			20	0.4	0.72	0.3	0.69
			10	0.2	0.36	0.2	0.49
Coastal	200	100	1000	3	30	0.9	9
			200	2	20	0.7	6
			100	1	10	0.5	3
			50	0.5	5	0.4	1.5
			20	0.2	2	0.2	0.6

Flood extent

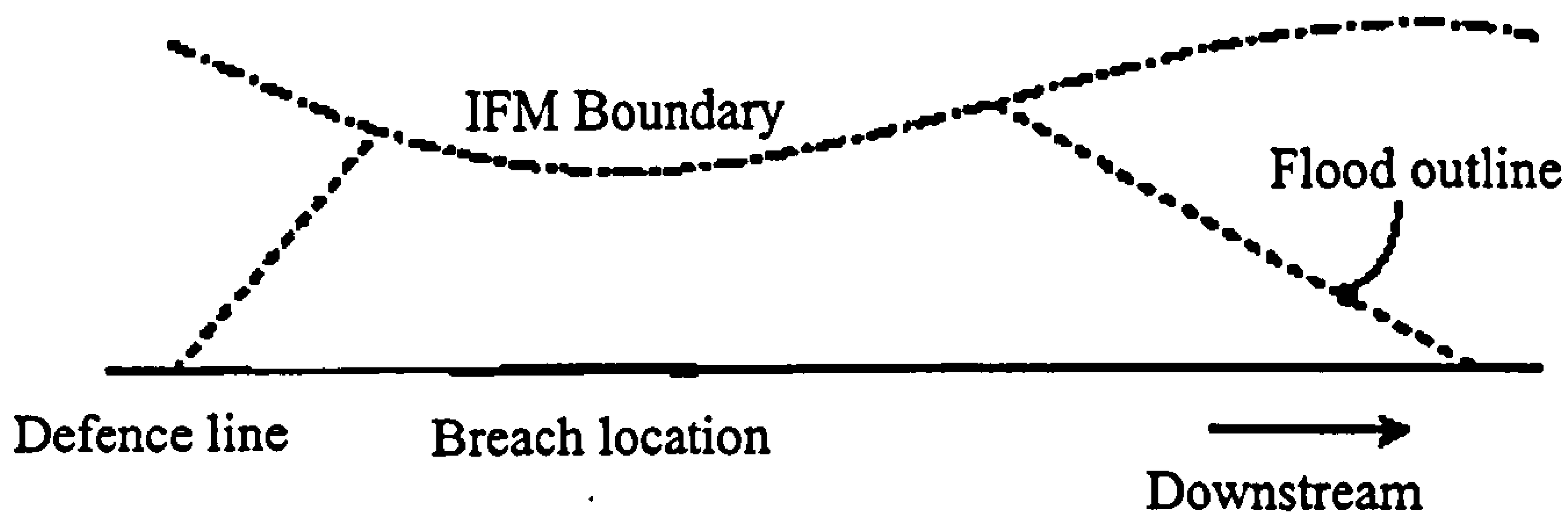
The flood outline is obtained from the flood volume, average flood depth and outline shape. A uniform flood depth, $d = 0.2$m is assumed. (Note that this depth is used only to establish the flood outline and is not further used to estimate the distribution of flood depths or flood damage). The floodplain is classified using a 1:50,000 digital terrain model and the Indicative Floodplain Maps (IFMs) as U-shaped for flat floodplains, V-shaped for steeply sloping narrow floodplains, or W-shaped for compound floodplains in which depths flood depths increase from the river and then reduce (Penning-Rowsell and Chatterton, 2000) as shown in Table F.4. U-shaped and coastal floodplains are assumed to have a semi-circular flood outline centred at the point of failure, with equal up and downstream flooding (Figure F.1 (a)). V-shaped floodplains are assigned a triangular flood outline, with greater downstream flooding (Figure F.1(b)). For all floodplains, when the flood outline reaches the extent of the Indicative Floodplain it is elongated upstream and downstream rather than allowing it to cross the floodplain boundary (Figure F.1 (c)).



(a) U-shaped and coastal floodplains



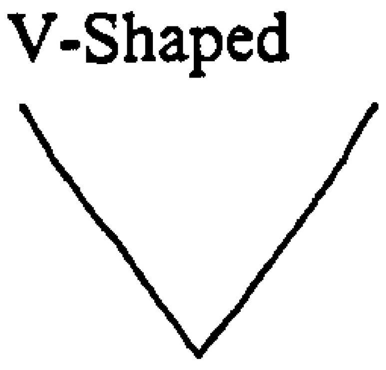
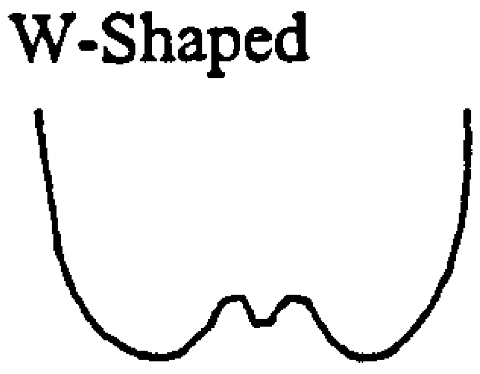
(*b*) V-shaped floodplains



(*c*) V-shaped floodplains where flood outline meets boundary of indicative floodplain map

Figure F.1 Assumed flood outlines for different valley types

Table F.4 Definition of floodplain types (Penning-Rowsell and Chatterton, 2000)

Geomorphology	**Floodplain width (m)**		
	Less than 250m (narrow)	250-500m (intermediate)	500-1000m (wide)
Floodplain shape	**Floodplain characteristics**		
U-Shaped 	Flat flood profile: Sloping to floodplain boundary		
V-Shaped 	Steeper flood profile: Largely restricted to narrow floodplains		
W-Shaped 	Compound profile: Depths rising from river and then falling		
Coastal	Flat flood profile		

Flood depth

Penning-Rowsell and Chatterton (2000) reviewed 70 flood scenarios (real and simulated) and generated estimates of flood depth at points across the floodplain in floods of a range of severities (Table F.5). These data were used to estimate flood depth at points between a failed defence and the floodplain boundary, in events of a given severity. Due to the limited number of analyses, some extreme estimates of flood depth distributed the mean and so the median value of depth is taken instead. The depth is assumed to decrease linearly, by factor *e*, with distance upstream and downstream of the failure location, where $e = 1$ at the failed defence and $e = 0$ at the predicted limit of the flood outline (Figure F.2). In the case of multiple defence failures resulting in the flood outlines overlapping, the factor *e* is aggregated to a maximum of 1. If a given point in the floodplain is predicted to be inundated in *k* different defence failure scenarios, each of which results in a flood depth $y_j, j = 1,\ldots, k$, with corresponding probability P_j then the probability of the flood depth Y exceeding some value y is given by:

$$P(Y \geq y) = \sum_{y_j \geq y} P_j \qquad \text{(F.9)}$$

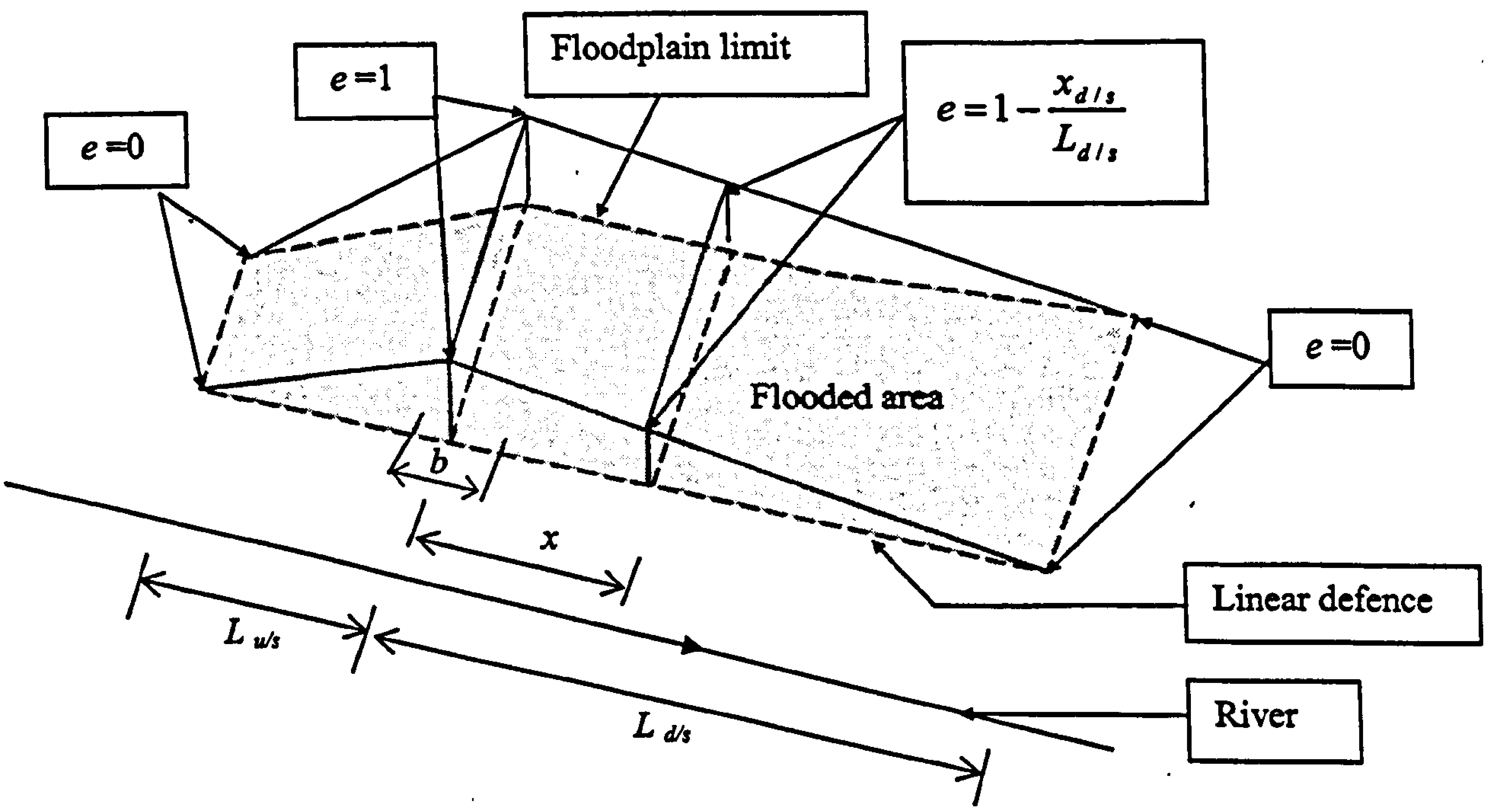


Figure F.2 How depth of flooding varies along river from the point of defence failure (e=1)

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Table F.5 Example of statistical data of flood depths (Penning-Rowsell and Chatterton, 2000)

Return Period Band		Percentile of Floodplain width																			
		0	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95
75-100	Max	2.98	2.87	2.31	2.38	2.81	2.9	2.71	2.51	2.99	2.89	2.81	2.98	2.83	2.31	1.93	2.3	2.03	2.6	2.06	0
	Median	0.9	0.87	1.23	0.89	1	1.05	0.8	0.99	1.15	1.1	0.71	0.86	1.93	1.84	1.54	1.54	2.03	1.55	1.38	0
	95%	2.61	2.49	2.19	2.3	2.55	2.84	2.63	2.35	2.75	2.63	2.45	2.7	2.62	2.22	1.88	2.2	1.03	2.5	1.99	0
	75%	1.43	1.47	1.68	1.44	2.24	1.73	1.92	1.62	1.99	1.66	1.68	1.84	1.99	1.94	1.68	1.78	2.03	2.08	1.72	0
	Sample	22	23	22	21	17	17	17	16	12	8	8	8	6	6	4	4	1	2	2	0
50-74	Max	2.98	2.65	2.11	2.49	2.58	2.68	2.83	2.83	2.69	2.74	2.72	2.68	2.53	2.01	1.63	2	1.73	2.6	1.76	0.21
	Median	0.62	0.67	0.59	0.55	0.94	0.84	0.66	0.64	0.6	0.69	0.6	0.6	1.14	1.13	0.5	0.9	0.94	1.4	1.08	0.21
	95%	2.46	2.1	1.76	2.09	2.25	2.48	2.39	2.35	2.41	2.15	2.65	2.44	2.33	1.93	1.54	1.88	1.65	2.48	1.69	0.21
	75%	1.15	0.93	1.06	0.91	1.59	1.21	1.46	1.39	1.62	1.07	1.51	1.59	1.71	1.69	1.2	1.4	1.34	2	1.42	0.21
	Sample	40	42	38	32	23	23	22	19	17	12	9	8	6	6	5	4	2	2	2	1
25-49	Max	2.6	2.65	2.28	2.65	1.9	1.1	1.56	1.09	1.38	2.4	2.4	1.83	1.56	1.53	0.34	0.31	0	0	0	0
	Median	0.52	0.57	0.57	0.44	0.49	0.65	0.51	0.55	0.49	0.79	0.53	0.33	0.75	0.96	0.34	0.31	0	0	0	0
	95%	1.24	1.4	1.48	2.33	1.42	1.04	1.35	1.04	1.25	2.04	2.13	1.62	1.48	1.47	0.34	0.31	0	0	0	0
	75%	0.9	0.84	0.77	0.77	0.91	0.89	0.85	0.74	0.97	0.93	1.05	0.76	1.16	1.24	0.34	0.31	0	0	0	0
	Sample	28	28	26	21	13	11	10	9	8	6	4	4	2	2	1	1	0	0	0	0
10-24	Max	2.12	2.07	1.64	1.98	2.31	2.25	2.55	2.55	2.41	2.46	2.44	2.4	2.25	1.73	1.35	1.72	1.45	2.45	1.48	-0.01
	Median	0.44	0.4	0.48	0.39	0.4	0.56	0.4	0.47	0.5	0.62	0.45	0.41	0.41	0.25	0.13	0.4	0.7	1.08	0.69	-0.01
	95%	1.9	1.92	1.49	1.91	2.03	1.98	2.33	1.77	1.9	2.32	2.36	2.17	2.04	1.62	1.19	1.59	1.37	2.31	1.4	-0.01
	75%	0.83	0.75	0.86	0.68	1.08	0.89	1.03	0.78	0.91	1.15	2.05	1.22	1.21	1.18	0.53	1.06	1.07	1.76	1.09	-0.01
	Sample	38	39	35	30	20	20	19	18	14	8	5	6	5	5	4	3	2	2	2	1
1-9	Max	1.73	1.78	1.28	1.47	1.65	1.45	1.88	1.23	0.99	0.59	0.15	0.05	-0.15	0	-0.11	0	0	0	0	0
	Median	0.33	0.25	0.23	0.24	0.25	0.29	0.25	0.07	0.22	0.25	0.25	-0.08	-0.15	0	-0.11	0	0	0	0	0
	95%	1.29	1.25	0.79	0.88	1.48	1.41	1.81	1.1	0.87	0.55	0.15	0.04	-0.15	0	-0.11	0	0	0	0	0
	75%	0.42	0.5	0.56	0.41	0.76	0.69	0.72	0.56	0.59	0.37	0.15	-0.01	-0.15	0	-0.11	0	0	0	0	0
	Sample	29	28	23	20	13	13	12	11	9	3	1	2	1	0	1	0	0	0	0	0

Appendix G

National flood risk assessment 2002

The National Flood Risk Assessment 2002 (NFRA) was a project designed to update the previous national assessment of flood risk (HR Wallingford *et al.*, 2000, Halcrow *et al.*, 2001) using the High Level Risk Assessment methodology described in Chapter 4 of this thesis. It should be noted that the author did not directly perform the national analysis, but as a key contributor towards the creation of the risk assessment methodology and the implementer of the case study for the river Parrett was a member of the advisory board for the project. Summary results of the NFRA 2002 are provided as a matter of interest to the reader and a reminder of the relevance and applicability of the research described in this thesis. Results of the NFRA 2002 (reproduced from HR Wallingford *et al.*, 2003) are summarised in Table G.1 to Table G.3 below.

The previous national scale flood risk assessment estimated the national assessment of flood risk to be £626 million per year (Halcrow *et al.*, 2001). Using the risk assessment methodology described in Chapter 4, this figure has now been calculated as being between £601 and £2,161 million per year with a best estimate of £1,060 million per year.

Expected annual damage results from the NFRA have been categorised into three risk bands. Figure G.1 shows the expected annual damage output from the NFRA for the Parrett catchment. Figure G.2 shows the annual likelihood of inundation, again categorised into three bands, and the probability of defences being breached or overtopped.

The two most likely causes of the difference between these estimates is the change in methodology and the difference between the depth-damage curves used. Although still limited by the availability of data on a national scale, the high level method provides a number of improvements over the previous national scale flood risk assessment by Halcrow *et al.* (2001) because:

- defence failure is calculated as a function of load rather than as a point value, thereby providing a complete overview of defence performance.
- the extent of flooding is estimated instead of assuming that a defence breach along a given reach floods all land behind this reach, thereby allowing defences to be associated with the parts of the floodplain they protect.

- multiple defence failures are considered, allowing the compound effects to be analysed, thereby considering all possible failure scenarios that can result in flooding of a given impact zone.
- the depth of flooding is estimated instead of assuming inundation of a percentage of the floodplain behind a reach, thereby providing more accurate damage estimation.

Table G.1 Results for flood risk at regional and national level

Region	Number of Properties at Risk (*k*)			Number of People at Risk (*k*)
	Residential	*Commercial*	*Agricultural (k hectares)*	
Anglian	292	22.6	571	701
Midlands	193	16.5	173	464
NorthEast	243	15.7	196	583
NorthWest	144	10.5	90	345
Southern	142	14.3	112	341
SouthWest	71.3	8.3	111	171
Thames	418	35.1	68	1000
Wales	102	8.2	107	244
National	**1,610**	**131**	**1,430**	**3,850**

Table G.2 Results for flood risk at regional and national level

Region	Value of Property at Risk (£ Billion)		Expected Annual Damage (£ million / year)		
	Residential	*Agricultural*	*Lower bound*	*Upper bound*	*Best estimate*
Anglian	40.1	3.04	106	370	184
Midlands	21.5	0.78	37	143	68
NorthEast	18.5	0.84	95	295	159
NorthWest	12.6	0.39	71	226	119
Southern	24.5	0.48	52	181	91
SouthWest	9.9	0.41	27	118	52
Thames	71.6	0.27	153	572	276
Wales	10.0	0.31	60	256	111
National	**208.7**	**6.52**	**601**	**2161**	**1060**

Table G.3 Distribution of Average Annual Damage at regional and national level

Region	% Area within Region for Given Expected Annual Damage (EAD)*		
	EAD < 1k / ha	**£1k / ha < EAD < £5k / ha**	**£5k / ha < EAD**
Anglian	96%	2%	1%
Midlands	93%	3%	1%
NorthEast	92%	4%	4%
NorthWest	88%	7%	4%
Southern	92%	5%	3%
SouthWest	95%	2%	2%
Thames	79%	11%	9%
Wales	93%	4%	3%
National	**93%**	**4%**	**2%**

(*Note*: All agricultural land categorised as <£1k/ha)

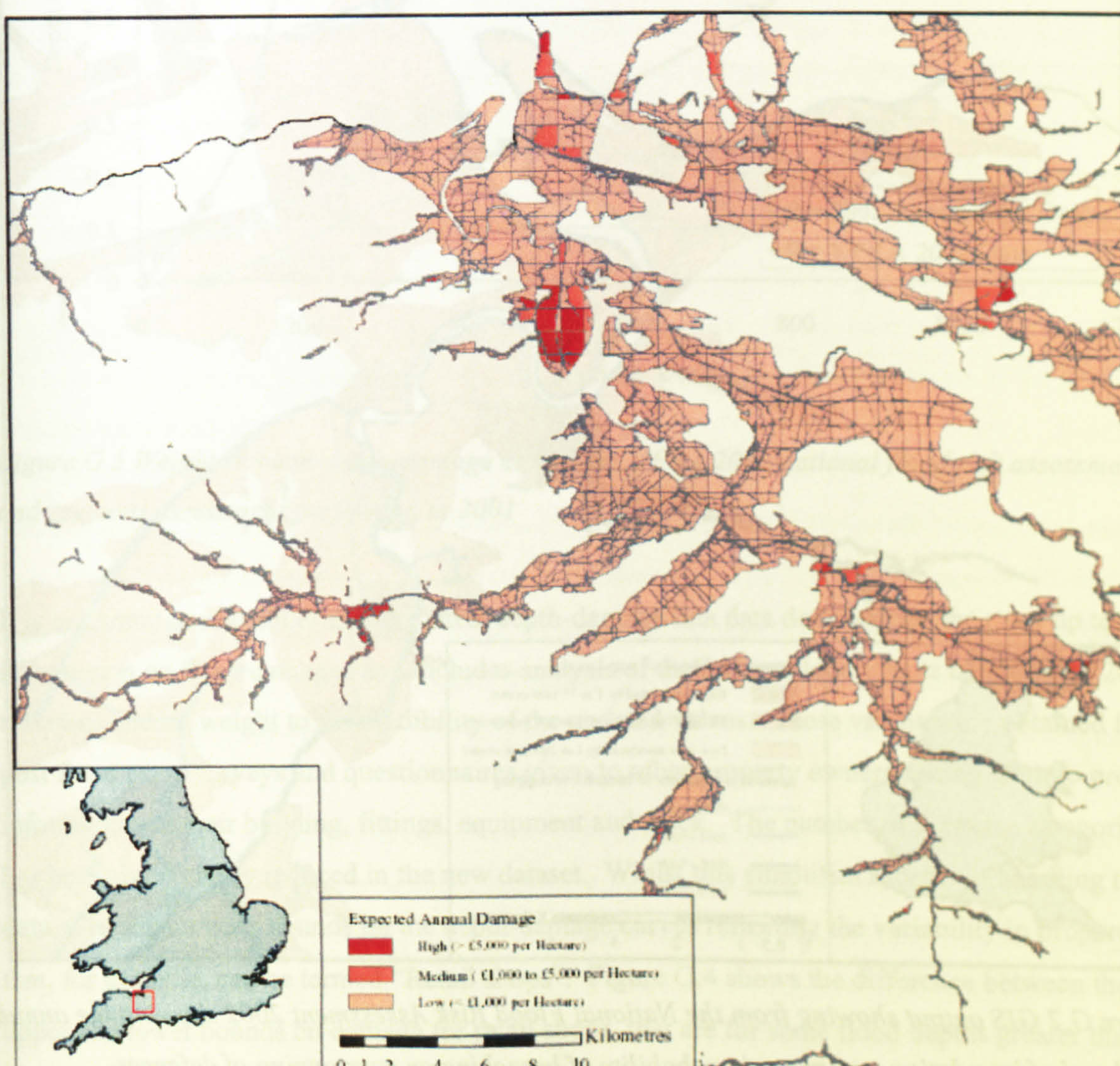

Figure G.1 GIS output from the National Flood Risk Assessment 2002 showing the expected annual damage for the Parrett catchment

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* A small percentage of the floodplain was disassociated from rivers and coastline and was therefore not evaluated.

The previous national assessment used the FLAIR dataset (Middlesex University, 1990), whereas the NFRA 2002 used data from the recently published Multi-Coloured Manual (Penning-Rowsell *et al.*, 2003). The FLAIR dataset was also used in the case study in Chapter 4 to allow comparison with the previous study. On average the damage values used have increased two or threefold using the latest datasets. Using the old methodology – but with the new depth-damage curves, the flood risk would be calculated to be over 100% of its original estimate (HR Wallingford *et al.*, 2003). This suggests that whilst the updated depth-damage dataset increases the overall estimate of flood risk, the improved high level risk assessment methodology acts to reduce the estimate.

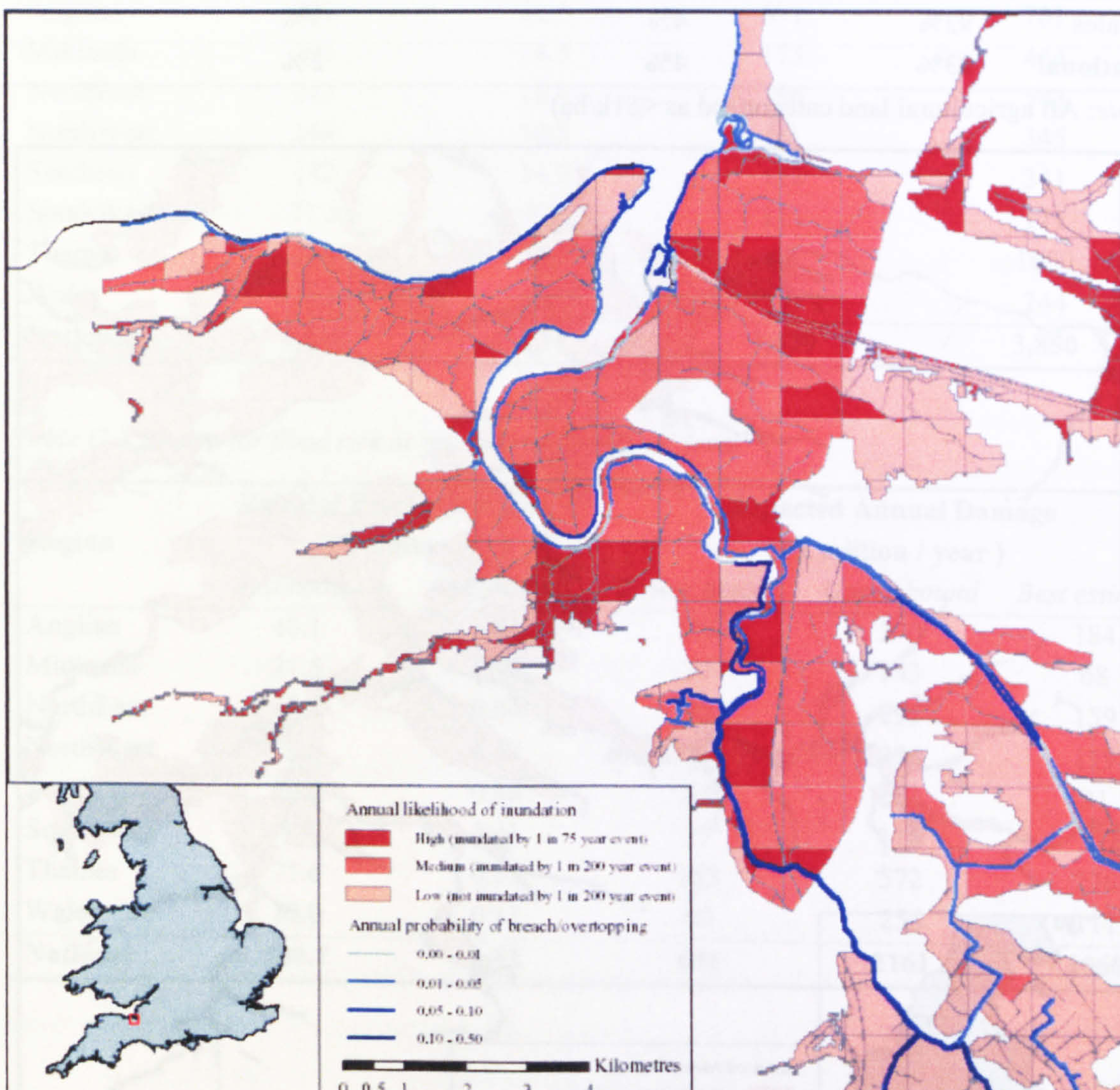

Figure G.2 GIS output showing from the National Flood Risk Assessment 2002 showing the annual likelihood of inundation and annual probability of breaching or overtopping of defences

The autumn 2000 floods in England and Wales resulted in damages (to property and agricultural land) of the order of £1billion (Penning-Rowsell *et al.*, 2002). This was caused by the wettest autumn on record and resulted in 10,000 properties being flooded at over 700 locations

(Environment Agency, 2001). It may therefore seem surprising that the national assessment of annual average damage is in the order of £1billion too. A lower figure might well be expected. A number of possible reasons for this being the case are now discussed.

As previously identified, the depth-damage data used (Penning-Rowsell *et al.*, 2003) was substantially higher than that used (Middlesex University, 1990) in the previous national assessment of flood risk. The difference between the two datasets can be seen in Figure G.3.

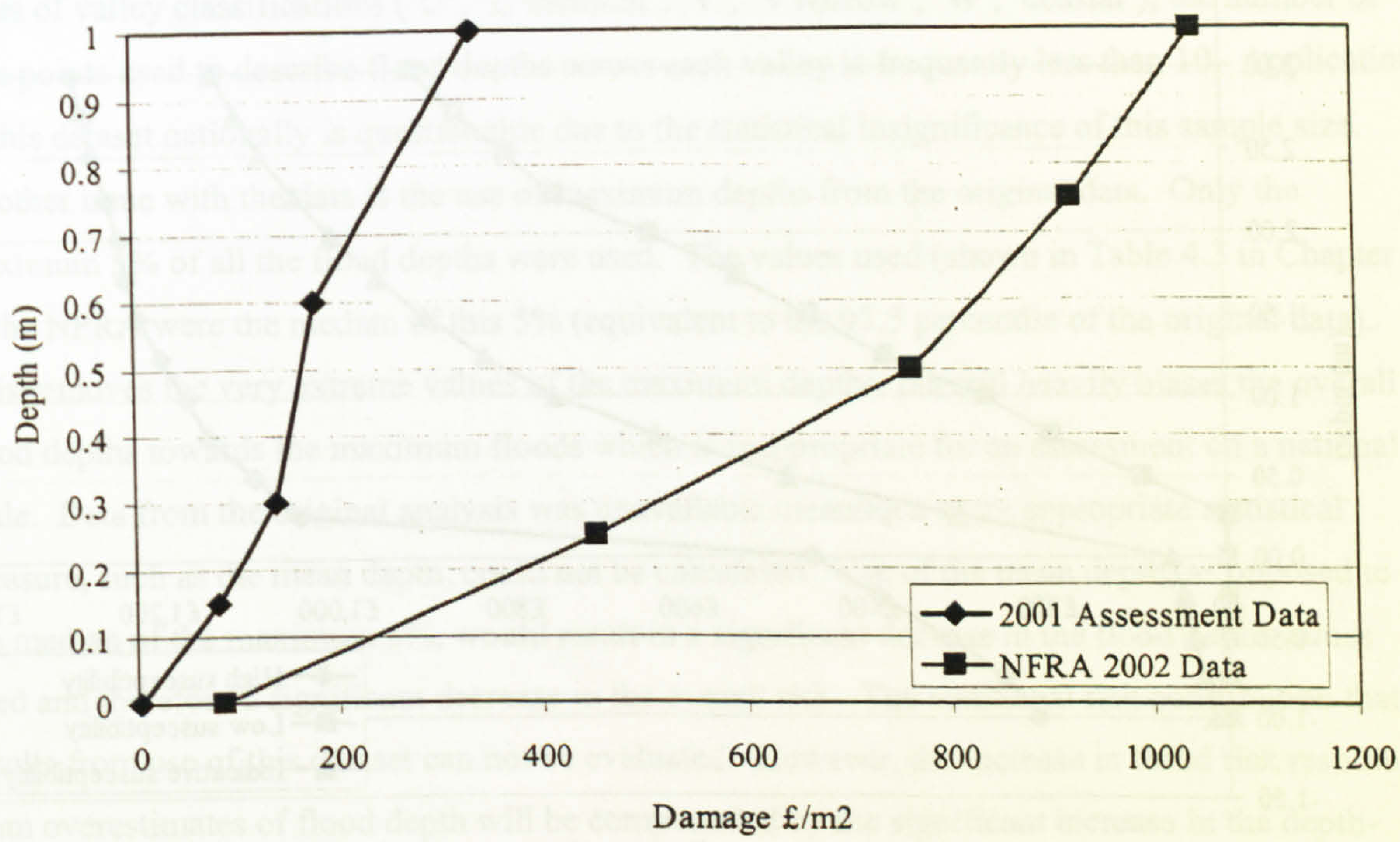

Figure G.3 Weighted mean depth-damage curves for NFRA 2002 national flood risk assessment and previous assessment performed in 2001

It is important to remember that the latest depth-damage this data does contain the most up to date information on flood damages and includes analysis of the serious flood events of 1998 and 2000, therefore adding weight to the credibility of the updated values. These values were obtained from post flood event surveys and questionnaires given to other property owners asking them to provide information on their building, fittings, equipment and stock. The number of property categories has been significantly reduced in the new dataset. Whilst this simplifies aspects of handling the data, it results in wide bounds on the depth-damage curves reflecting the variability in properties that, for example, can be termed "Retail shops". Figure G.4 shows the difference between the upper and lower bounds on damage for retail shops, that are for some flood depths greater than £1000/m^2 apart.

Inaccuracies in the SOP and condition grade data also affect the results of the risk asset management. These errors were also identified by Halcrow *et al.* (2001) in the previous national assessment of flood risk. On a national scale it is likely that many of these local errors will cancel themselves out. The fragility curve is based on only limited analysis and consequently the reliance on expert judgements may be identified as a source of uncertainty. However, the use of fragility curves provides a significant improvement over previous risk assessment methods that define the probability of failure as being constant over all loadings. This is because the fragility curve acts to reduce the contribution towards the flood risk from the less likely events.

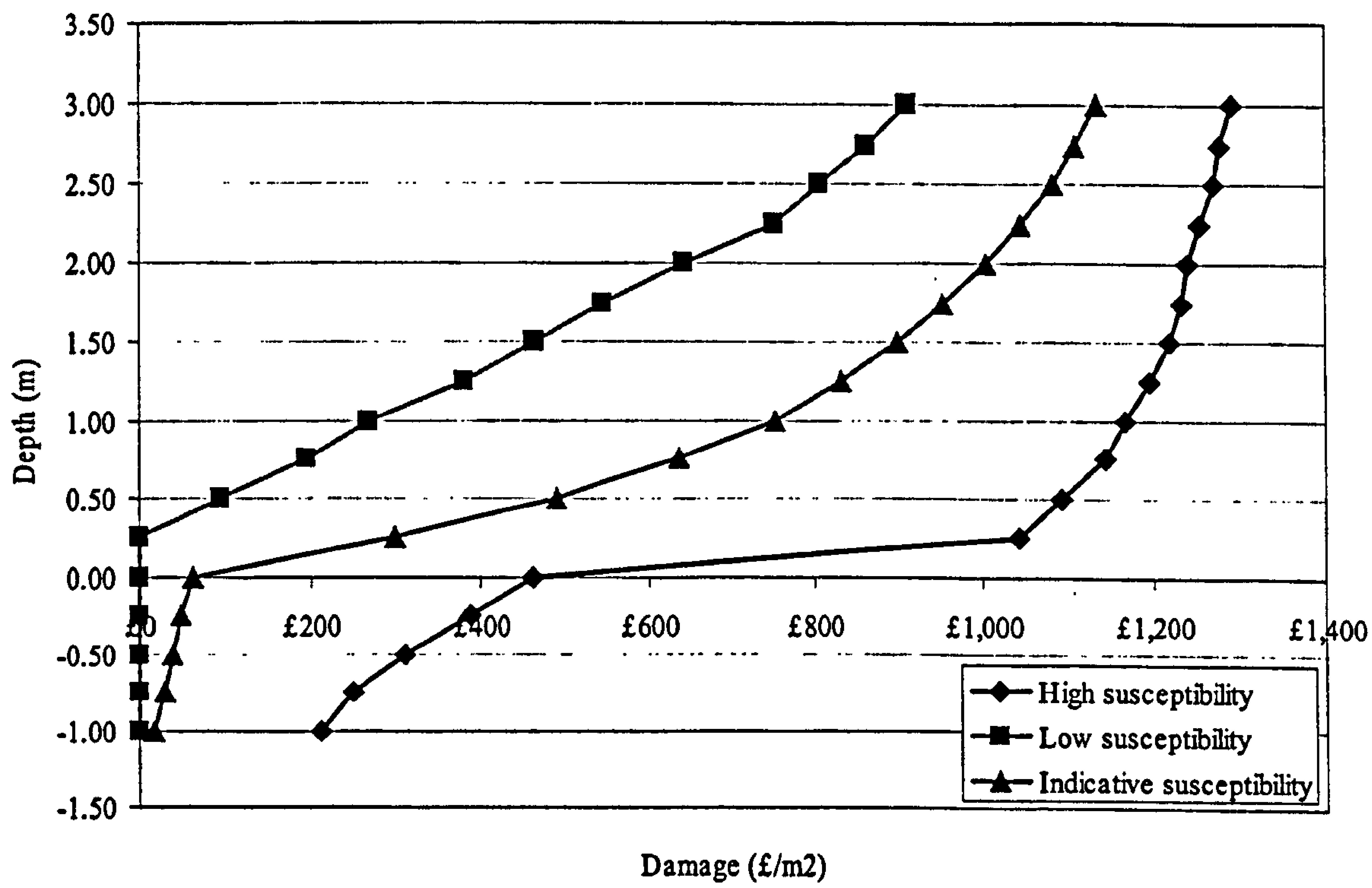

Figure G.4 Depth-damage curve for a "Retail shop", showing lower, upper and indicative damage bounds for fluvial flooding (Penning-Rowsell et al., 2002)

The source of uncertainty most likely to have resulted in a flood risk assessment that is too high is related to the inundation modelling. The parametric modelling equations, whilst initially based on rules of thumb and expert judgement, have been calibrated to reduce the associated uncertainty.

During the study it was recognised that the Indicative Floodplain Map (IFM) contained flaws. Areas of floodplain that are disassociated from watercourses and coastline were ignored in the analysis. Despite this, areas of high ground encapsulated within the IFM are not recognised. This error is particularly significant in some cities where enormous engineering channels, capable of conveying the 100 year flood without raised embankments are identified as having an associated 100 year floodplain. Inaccuracies have also been recognised in other studies (Mylius, 2003). Errors in the IFM result from a number of reasons. The IFM was constructed from a number of different sources that include the surveys done as a result of Section 105 of the Water Resources Act, other modelling studies, previous maximum recorded flood outlines and expert judgement. Inaccuracies in each of the datasets are likely to be compounded when they are aggregated. Due to insufficient data no hydrodynamic modelling could be undertaken to verify the outline or quantify the uncertainty associated with using it.

The IFM is not the only source of uncertainty associated with the data used in the inundation modelling. The most likely source of inaccuracy is the use of statistical flood data (Penning-Rowsell and Chatterton, 2000) to estimate flood depths across the floodplain. This data was constructed from approximately 70 real and simulated flood events. However, with 6 different types of valley classifications ('U', 'U medium', 'V', 'V narrow', 'W', 'coastal'), the number of data points used to describe flood depths across each valley is frequently less than 10. Application of this dataset nationally is questionable due to the statistical insignificance of this sample size. Another issue with the data is the use of maximum depths from the original data. Only the maximum 5% of all the flood depths were used. The values used (shown in Table 4.3 in Chapter 4) in the NFRA were the median of this 5% (equivalent to the 97.5 percentile of the original data). This removes the very extreme values of the maximum depths, but still heavily biases the overall flood depths towards the maximum floods which is inappropriate for an assessment on a national scale. Data from the original analysis was unavailable meaning a more appropriate statistical measure, such as the mean depth, could not be calculated. Use of the mean depth, as opposed to the median of the maximum 5%, would result in a significant decease in the flood depth values used and therefore a significant decrease in the overall risk. The additional risk contribution that results from use of this dataset can not be evaluated. However, the increase in flood risk resulting from overestimates of flood depth will be compounded by the significant increase in the depth-damage relationships.

Despite the fact that over 700 locations were flooded in 2000, only 20 locations contained more than 100 properties and in no single location were more than 230 properties flooded. No major city was seriously inundated. Therefore, whilst the floods were undoubtedly serious in some respects, their localisation meant that there is potential for significantly greater consequences given suitable conditions. This demonstrates the spatial complexity of the problem that flood risk managers face.

Human intervention is not included in this risk assessment and this has potential to significantly reduce flood risk. Emergency repair, for example, may make a significant contribution to reducing the frequency of breaching. During the 1995 fluvial floods in the Netherlands a number of dykes were heavily reinforced and it is very likely that as a result of this there were no breaches and therefore considerably reduced damages (Langemheem, 2002). Successful flood warning dissemination that results in an appropriate response from floodplain residents can also significantly reduce the damages (*eg.* by moving valuables) associated with a flood event (Figure G.5).

A number of points raised in the preceding sections have identified a number of possible sources of error in making a risk assessment on a national scale. The influence of some of these factors on the national flood risk assessment will remain unquantified. However, this does not render it useless. The national flood risk assessment provides a consistent and transparent tool with which to estimate the expected annual economic consequences of flooding. The methodology is a step towards a full probabilistic, process-based assessment of national flood risk. It can be used in national policy analysis by testing scenarios of changed flood frequency, investment in flood defences or floodplain occupancy. It also provides a starting point for more detailed analysis.

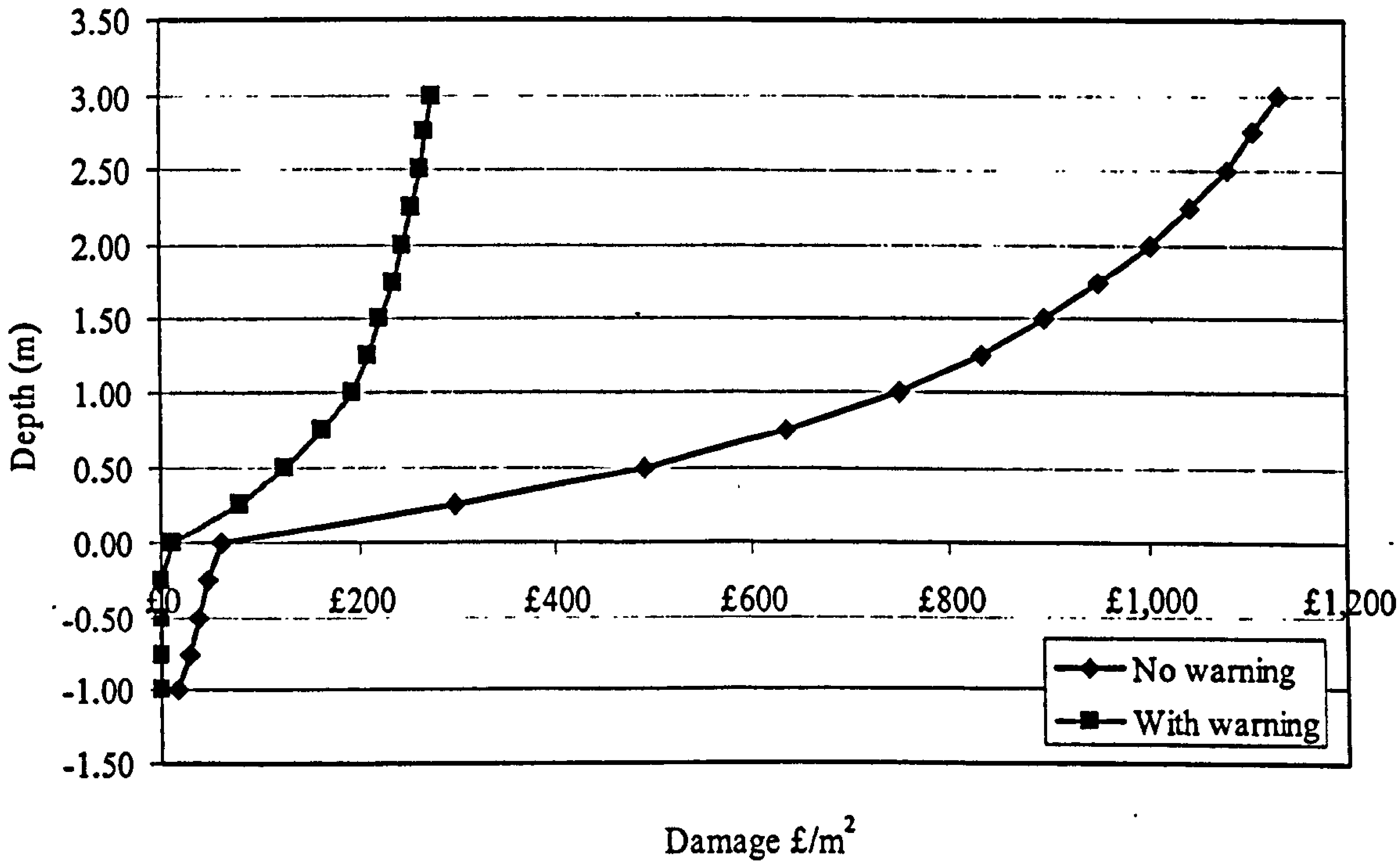

Figure G.5 Comparison between indicative depth-damage curves for a retail shop that receives a flood warning and a shop that does not receive a warning (Penning-Rowsell et al., 2003)

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Appendix H

Uncertain inference using interval probability theory

This appendix explains the evidence propagation calculation that is used by the PERIMETA software. The methodology used for evidence propagation was originally implemented by Hall *et al.* (1998). This was improved by Hall (1999) and recent developments have allowed the logical inference algorithm to be simplified. For simplicity, an example with two sub-systems propagating evidence to one super-system is used. The theory behind the calculation is first introduced and subsequently followed by an example.

In the following sections, E, is a proposition that provides information about a hypothesis, H. The evidence of performance is used to judge the belief in the proposition, E. The hypothesis relates to the super-system of the process model and the propositions relates to the sub-systems of the process model. The interval probability is defined as $P(E) \in [S_n(E), S_p(E)]$ where $S_n(E)$ represents the extent to which it is certainly believed that the proposition, E, is dependable and $1\text{-}S_p(E)$ represents the extent to which the evidence is certainly not dependable and the value $S_p(E)\text{-}S_n(E)$ represents the uncertainty of belief in the dependability of E.

H.1. COMPOUND PROPOSITIONS

H.1.1. Classical probability theory

Consider two propositions, E_1 and E_2 in which $P(E_1) = p$ and $P(E_2) = q$. p and q are measures on the interval [0,1] and do not necessarily add to unity. Figure H.1 shows how probability assignments, m_i, are distributed over $E_1 \times E_2$.

	E_1	$\overline{E_1}$	
E_2	m_1	m_2	$\Sigma=q$
$\overline{E_2}$	m_3	m_4	
	$\Sigma=p$		

Figure H.1 Representation of a compound proposition using classical probability theory

Therefore:

- $p=m_1+m_3=P(E_1)$,
- $q=m_2+m_4=P(E_2)$, and,
- $\sum m_i = 1$.

This problem has one degree of freedom and therefore one further piece of information is required about the relationship between E_1 and E_2 in order to establish values for m_i. Knowledge of p and q allows upper and lower bounds on the values of m_i to be allocated. For example if p=0.3 and q=0.6:

		E_1 0.3	$\overline{E_1}$ 0.7
E_2	0.6	m_1 [0.0,0.3]	m_2 [0.3, 0.6]
$\overline{E_2}$	0.4	m_3 [0.0, 0.3]	m_4 [0.1, 0.4]

Figure H.2 Bounds on probabilities with one unconstrained degree of freedom

H.1.2. Interval probability theory

Consider the two interval propositions E_1 and E_2, such that $P(E) \in [S_n(E), S_p(E)]$ with probability assignments, m_{ij}, to $E_1 \times E_2$ as shown in Figure H.3.

	$S_n(E_2)$ $\equiv E_2$	$1-S_p(E_2)$ $\equiv \overline{E_2}$	$S_p(E_2)-S_n(E_2)$ $\equiv E_{2U}$
$S_n(E_1)$ $\equiv E_1$	m_{11}	m_{12}	m_{13}
$1-S_p(E_1)$ $\equiv \overline{E_1}$	m_{21}	m_{22}	m_{23}
$S_p(E_1)-S_n(E_1)$ $\equiv E_{1U}$	m_{31}	m_{32}	m_{33}

Figure H.3 Representation of compound propositions using interval probability theory

The dependency between the two propositions is defined as the interval $[\rho_l, \rho_u]$. Referring back to Section 6.3.3:

$$S_n(E_1 \cap E_2) = \rho_l.\min(S_n(E_1), S_n(E_2)) \tag{H.1}$$

$$S_p(E_1 \cap E_2) = \rho_u.\min(S_p(E_1), S_p(E_2)) \tag{H.2}$$

$$S_n(E_1 \cup E_2) = S_n(E_1) + S_n(E_2) - \rho_l.\min(S_n(E_1), S_n(E_2)) \tag{H.3}$$

$$S_p(E_1 \cup E_2) = S_p(E_1) + S_p(E_2) - \rho_u.\min(S_p(E_1), S_p(E_2)). \tag{H.4}$$

In terms of interval probabilities:

$$P(E_1 \cap E_2)=[m_{11},\ m_{11}+m_{13}+m_{31}+m_{33}] \tag{H.5}$$

$$P(E_1 \cap \overline{E_2})=[m_{12},\ m_{12}+m_{13}+m_{32}+m_{33}] \tag{H.6}$$

$$P(\overline{E_1} \cap E_2)=[m_{21},\ m_{21}+m_{23}+m_{31}+m_{33}] \tag{H.7}$$

$$P(\overline{E_1} \cap \overline{E_2})=[m_{22},\ m_{22}+m_{23}+m_{32}+m_{33}] \tag{H.8}$$

And the bounds on $P(E_1)$ and $P(E_2)$ are:

$$P(E_1)=[m_{11}+m_{12}+m_{13},\ m_{11}+m_{12}+m_{13}+m_{31}+m_{32}+m_{33}] \tag{H.9}$$

$$P(E_2)=[m_{11}+m_{13}+m_{21},\ m_{11}+m_{13}+m_{21}+m_{23}+m_{31}+m_{33}] \tag{H.10}$$

The values of m_{ij} on the interval [0, 1] are constrained such that:

$$S_n(E_1)=m_{11}+m_{12}+m_{13} \tag{H.11}$$

$$1-S_p(E_1)=m_{21}+m_{22}+m_{23} \tag{H.12}$$

$$S_n(E_2)=m_{11}+m_{21}+m_{31} \tag{H.13}$$

$$1-S_p(E_2)=m_{12}+m_{22}+m_{32} \tag{H.14}$$

$$\sum_{j=1}^{n}\sum_{i=1}^{n} m_{ij}=1 \tag{H.15}$$

Therefore, from Equations H.1 and H.5:

$$m_{11}=\rho_l \times \min(S_n(E_1), S_n(E_2)) \tag{H.16}$$

and from Equation H.8:

$$m_{22}=S_n(\overline{E_1} \cap \overline{E_2})=1-S_p(E_1 \cup E_2) \tag{H.17}$$

so from Equation H.4:

$$m_{22}=1-S_p(E_1)-Sp(E_2)+\rho_u \times \min(S_p(E_1), S_p(E_2)) \tag{H.18}$$

$P(E_1 \cap E_2)$ and $P(\overline{E_1} \cap \overline{E_2})$ are uniquely defined, however unique intervals under all values of $P(E_1)$, $P(E_2)$ and ρ for $P(\overline{E_1} \cap E_2)$ and $P(E_1 \cap \overline{E_2})$ cannot be found with the restraints of Equations H.11 to H.18. To obtain these values would require knowledge of the dependency between E_1 and $\overline{E_2}$ and between $\overline{E_1}$ and E_2, therefore a family of permissible values is calculated (Figure H.6) where ρ=1 indicates that $E_1 \subset E_2$ (*i.e.* E_1 and E_2 are nested propositions) whilst

$$\rho=\max(P(E_1), P(E_2)) \tag{H.19}$$

if they are independent. The minimum value of ρ is given by

$$\rho=\max\left[\frac{P(E_1)+P(E_2)-1}{\min(P(E_1),P(E_2))},\ 0\right] \tag{H.20}$$

where $\rho=0$ indicates that E_1 and E_2 are disjoint or mutually exclusive such that $E_1.E_2=\emptyset$.

This approach to obtaining the assignments can be extended to three or more propositions. For three propositions the compound proposition figure would take the form of a cube, for n propositions the figures will occupy n-dimensional space. A problem with n propositions will have 3^n degrees of freedom. Assigning an interval probability using this methodology provides $2n$ constraints, the dependency measures will provide an additional $n!/(n-2)!$ constraints, with a final constraint provided by the probabilities summing to unity. Thus for n propositions there will be:

$$3^n - 2n - \frac{n!}{(n-2)!} - 1$$

degrees of freedom (this compares to 2^n-n-1 degrees for compound propositions using classical probability theory). This provides a cogent motive for implementing the methodology using software and describing only a small evidence propagation problem in detail.

H.2. LOGICAL INFERENCE

Having established a method for combining probabilities, the relationship between the propositions, $E_1, E_2 \ldots E_n$ about the performance of sub-systems $1,2\ldots n$ and some hypothesis, H, about the performance of their super-system needs to be addressed. To establish the support, $P(H)$, on the basis of the propositions, $P(E)$ and the relationship between E and H need to be established. This relationship is defined by the conditional measures $P(H|E)$ and $P(H\,|\,\overline{E})$ which can be obtained using the theorem of total probability (see Section 3.2.4). Referring back to Section 6.3.6, for a single proposition Dubois and Prade (1990) adapted the total probability theorem for interval probability to show the bounds on H, S_n and S_p can be calculated using:

$$S_n(H) = S_n(H\,|\,E)S_n(E) + S_n(H\,|\,\overline{E})(1 - S_n(E)) \;;\; S_n(H\,|\,E) \ge S_n(H\,|\,\overline{E}),$$

$$S_n(H) = S_n(H\,|\,E)S_p(E) + S_n(H\,|\,\overline{E})(1 - S_p(E)) \text{ ; otherwise} \tag{H.21}$$

and

$$S_p(H) = S_p(H\,|\,E)S_p(E) + S_p(H\,|\,\overline{E})(1 - S_p(E)) \;;\; S_p(H\,|\,E) \ge S_p(H\,|\,\overline{E}),$$

$$S_p(H) = S_p(H\,|\,E)S_n(E) + S_p(H\,|\,\overline{E})(1 - S_n(E)) \text{ ; otherwise} \tag{H.22}$$

In the case where there are two propositions, E_1 and E_2, the total probability theorem dictates that:

$$\begin{aligned} P(H) = {} & P(H\,|\,E_1 \cap E_2).P(E_1 \cap E_2) + P(H\,|\,\overline{E_1} \cap \overline{E_2}).P(\overline{E_1} \cap \overline{E_2}) + \\ & P(H\,|\,\overline{E_1} \cap E_2).P(\overline{E_1} \cap E_2) + P(H\,|\,E_1 \cap \overline{E_2}).P(E_1 \cap \overline{E_2}) \end{aligned} \tag{H.23}$$

where $P(H\,|\,E_1 \cap E_2)$, $P(H\,|\,\overline{E_1} \cap \overline{E_2})$, $P(H\,|\,\overline{E_1} \cap E_2)$ and $P(H\,|\,E_1 \cap \overline{E_2})$ define the relationship between H and E_1 and E_2. This is illustrated by the Venn diagram shown in Figure H.4.

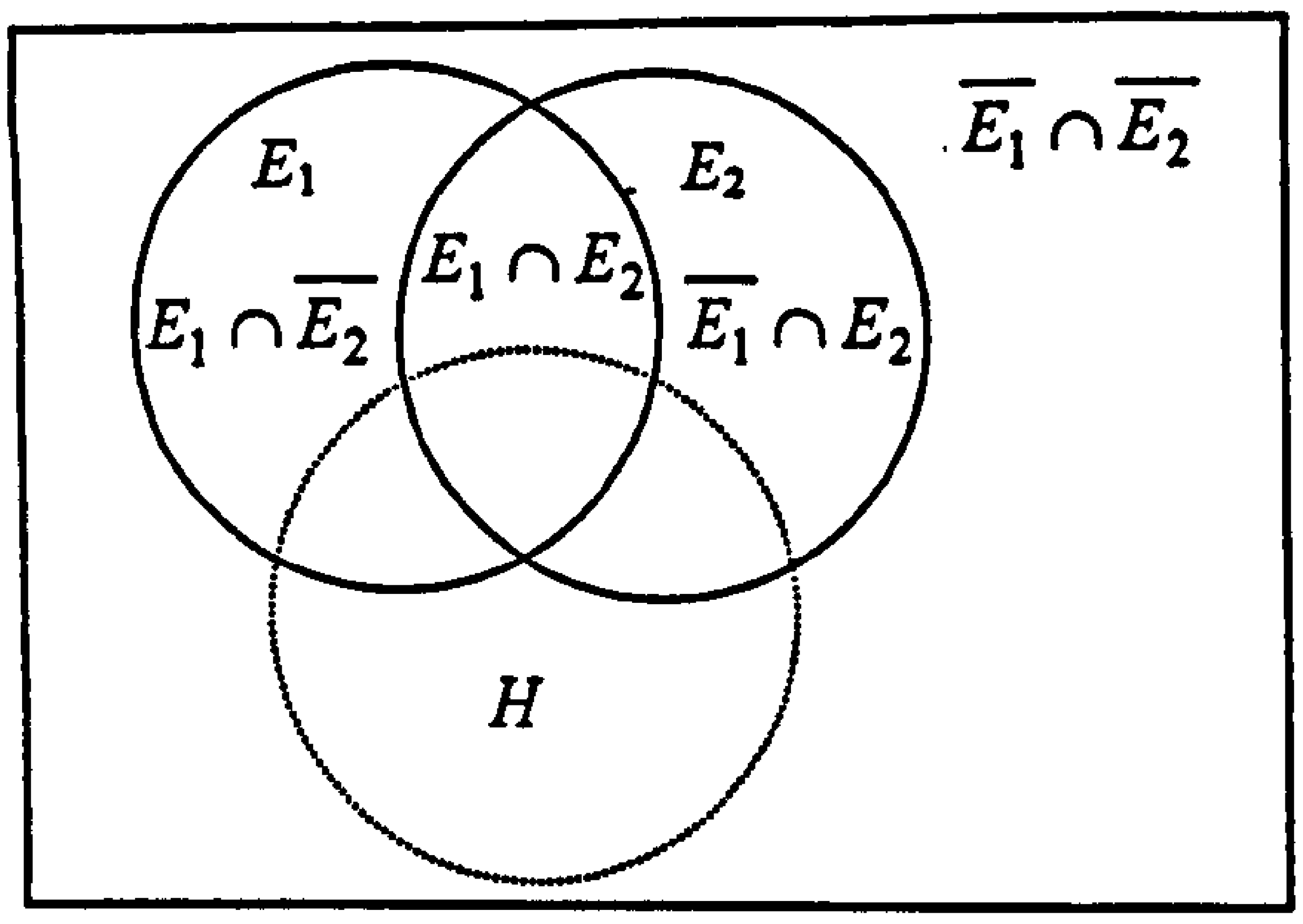

Figure H.4 Relationship between H and two propositions E_1 and E_2

So, for example, if E_1 and E_2 are both necessary conditions for H then $P(H \mid \overline{E_1} \cap E_2) = P(H \mid E_1 \cap \overline{E_2}) = P(H \mid \overline{E_1} \cap \overline{E_2}) = 0$, meaning that $P(H) = P(H \mid E_1 \cap E_2).P(E_1 \cap E_2)$. If both propositions are necessary and sufficient for H, then $P(H|E_1 \cap E_2)=1$, so $P(H)=P(E_1 \cap E_2)$. It is not usually the case that hypotheses are entirely sufficient or necessary. However, Equation H.23 can be re-written as:

$$P(H) = \begin{Bmatrix} P(H \mid E_2).P(E_2) + P(H \mid E_1).P(E_1) - \\ P(H \mid E_1 \cap E_2).P(E_1 \cap E_2) + P(H \mid \overline{E_1} \cap \overline{E_2}).P(\overline{E_1} \cap \overline{E_2}) \end{Bmatrix} \quad \text{(H.24)}$$

The conditional probabilities can be defined by the modeller (Section H.3). Table H.1 shows the assignments that potentially contribute to the probabilities of three of the four subsets in Equation H.24. The subset $P(E_1 \cap E_2)$ does not need to be included in Table H.1. The following methodology ensures that the assignments m_{11}, m_{31}, m_{13}, m_{33} which would contribute to this probability are not double counted, thereby negating the need to subtract them.

Table H.1 Assignments that contribute to the upper bound of the probabilities

	m_{11}	m_{12}	m_{13}	m_{21}	m_{22}	m_{23}	m_{31}	m_{32}	m_{33}	**Corresponding Conditional probability (C_i)**
$P(E_1)$	1	1	1	0	0	0	1	1	1	$P(H|E_1)$
$P(E_2)$	1	0	1	1	0	1	1	0	1	$P(H|E_2)$
$P(\overline{E_1} \cap \overline{E_2})$	0	0	0	0	1	1	0	1	1	$[S_n(H \mid \overline{E_1} \cap \overline{E_2}),$ $S_p(H \mid \overline{E_1} \cap \overline{E_2})]$

Upper and lower bounds on S_n and S_p can now be established. The minimum of $S_n(H)$ will occur when the largest combination of mass assignments are applied to the smallest conditional probability. This procedure is formalised as follows:

(1) Use $S_n(H \mid \overline{E_1} \cap \overline{E_2})$ as the conditional probability related to $P(\overline{E_1} \cap \overline{E_2})$.

(2) Order the conditional probabilities such that $C_1 < C_2 < C_3$.

(3) For C_1, sum all the corresponding values of m_{ij} from Table H.1 to obtain M_1.

(4) For C_2, sum all values of m_{ij} from Table H.1 that have not been summed previously (M_2).

(5) For C_3, sum the remaining values of m_{ij} from Table H.1 (M_3).

(6) Subject to restraints defined in Figure H.6:

$$S_n(H) = \min_{m_{12}, m_{21}} \left[C_1 M_1 + C_2 M_2 + C_3 M_3\right] \tag{H.25}$$

To calculate $S_p(H)$:

(1) Use $S_p(H \mid \overline{E_1} \cap \overline{E_2})$ as the conditional probability related to $P(\overline{E_1} \cap \overline{E_2})$.

(2) Order the conditional probabilities such that $C_1<C_2<C_3$.

(3) For C_3, sum all the corresponding values of m_{ij} from Table H.1 to obtain M_3.

(4) For C_2, sum all values of m_{ij} that have not been summed previously (M_2).

(5) For C_1, sum the remaining values of m_{ij} (M_1).

(6) Subject to restraints defined in Figure H.6:

$$S_p(H) = \max_{m_{12}, m_{21}} \left[C_1 M_1 + C_2 M_2 + C_3 M_3\right] \tag{H.26}$$

H.3. Defining the relationship between propositions and the hypothesis

In terms of software implementation, for ease of use, the model developer is asked to enter three parameters; dependency, sufficiency and necessity. These define the relationship between a proposition, E, and the hypothesis, H. The relationship between interval probability theory and these parameters is described below.

H.3.1. Dependency between propositions

Dependency is first defined by the *dependency parameter* on a scale of [-1, 1]. A linear transformation converts this to the actual dependency, ρ, on a scale [0, 1] defined by three points in Table H.2 with points in between linearly extrapolated. This is a convenient means of exploring different dependence relationships when the exact nature is uncertain

Table H.2 The relationship between the dependency parameter and ρ

	Mutually exclusive	Independent	Dependent
ρ	0	$\max(P(E_1), P(E_2))$	1
Dependency parameter	-1	0	1

H.3.2. Necessity and sufficiency

As defined in Chapter 6, the *sufficiency*, *S*, is a measure of the influence that a given sub-system has on the performance of its parent or super-system, and, the *necessity*, *N*, is a measure of the extent to which failure (non-performance) of a sub-system will cause failure (non-performance) of its parent super-system. A necessity and sufficiency value is defined for each sub-process. The sufficiency value is directly mapped to the conditional probability:

$$P(H|E_i) = \mathrm{suf}(E_i) \tag{H.27}$$

The necessity value is used to approximate the conditional probabilities of the negation of all the propositions $P(H \mid \overline{E_1} \cap \overline{E_2} \cap ... \overline{E_n})$:

$$S_n(H \mid \overline{E_1} \cap \overline{E_2} \cap ... \overline{E_n}) = \frac{1}{2}\left(0 + \min_{i=1...n}[1 - \text{nec}(E_i)]\right) \quad \text{(H.28)}$$

$$S_p(H \mid \overline{E_1} \cap \overline{E_2} \cap ... \overline{E_n}) = \frac{1}{2}\left(1 + \max_{i=1...n}[1 - \text{nec}(E_i)]\right) \quad \text{(H.29)}$$

These heuristic rules were defined by averaging two intervals. The first interval is totally uncertain [0, 1] and reflects the fact that the exact relationship between the necessities of the propositions and the conditional probabilities is unknown. The second interval is the negation of minimum and maximum necessities of the propositions, [min(1-nec(E_1)), max(1-nec(E_2))], and uses the user-defined information on necessity to constrain the conditional probability of negation. This relationship between necessity and conditional probability was validated through model testing.

H.4. EXAMPLE

The example model is shown in Figure H.5 and the variables that can be defined by the modeller have been assigned the values shown in Table H.3. It can be seen from Figure H.5 that the bounds for the super-system are calculated as $P(H) \in [0.36, 0.73]$.

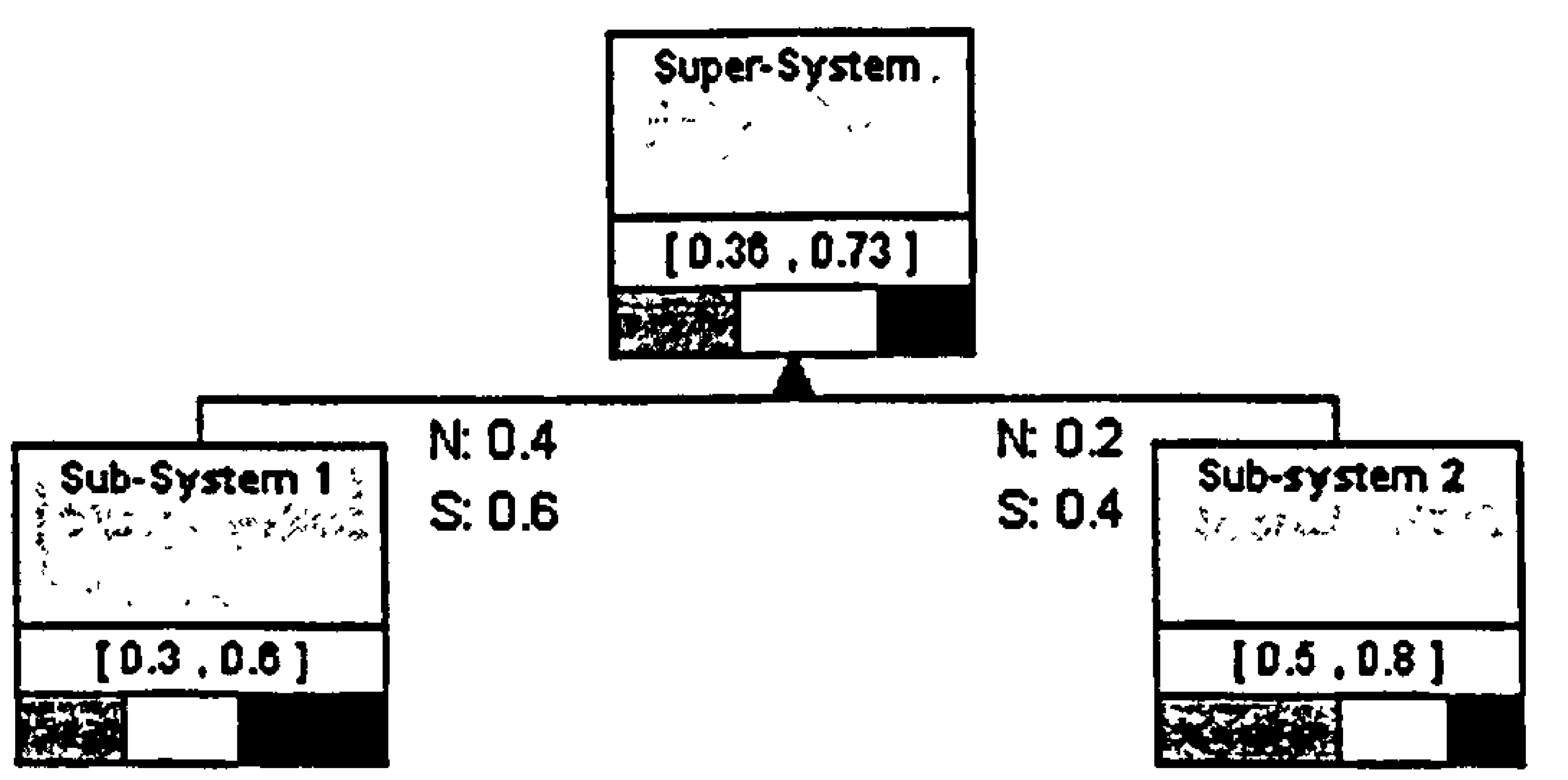

Figure H.5 Figure of merits for two sub-systems and their super-system

Table H.3 The input values for the example problem shown in Figure H.5

Sub-system 1		Sub-system 2	
$S_n(E_1)$	0.3	$S_n(E_2)$	0.5
$S_p(E_1)$	0.6	$S_p(E_2)$	0.8
Necessity (N_1)	0.4	Necessity (N_2)	0.2
Sufficiency (S_1)	0.6	Sufficiency (S_2)	0.4
Dependency	0.5	Dependency	0.5

The value of ρ assuming independence between the sub-systems is calculated as [0.5, 0.8]. The actual value of ρ (calculated using the user-defined dependency value of 0.5) is therefore [0.75, 0.9]. This enables the assignments of the compound proposition to be established as shown in Figure H.6.

	E_2 0.5	$\overline{E_2}$ 0.2	E_{2U} 0.3
E_1 0.3	0.23	m_{12}	0.07-m_{12}
$\overline{E_1}$ 0.4	m_{21}	0.14	0.26 -m_{21}
E_{1U} 0.3	0.27-m_{21}	0.06-m_{12}	m_{12}+ m_{21}- 0.03

subject to: $0 \le m_{12} \le 0.06$
$0 \le m_{21} \le 0.26$
$0.03 \le m_{21} + m_{12}$

Figure H.6 Compound proposition $P(E_1) \in [0.3, 0.6]$, $P(E_2) \in [0.5, 0.8]$, $\rho \in [0.75, 0.9]$ defined by dependency parameter of 0.5

Following the steps outlined previously for this example:

(1) $P(H \mid \overline{E_1} \cap \overline{E_2})$ is calculated as [0.3, 0.9], take $S_n(H \mid \overline{E_1} \cap \overline{E_2})$=0.3

(2) The conditional probabilities are ordered: C_1=0.3 ($P(H \mid \overline{E_1} \cap \overline{E_2})$) < C_2=0.4 ($P(H|E_2)$) < C_3=0.6 ($P(H|E_1)$)

(3) $M_1 = m_{22}+m_{23}+m_{32}+m_{33} = 0.43$

(4) $M_2 = m_{11}+m_{13}+m_{21}+m_{31} = 0.57-m_{12}$

(5) $M_3 = m_{12}$

(6) $S_n(H)$=min[0.3×0.43+0.4×(0.57-m_{12})+0.6×m_{12}]=0.36 as calculated by the software, this occurs when m_{12}=0 and $0.03 \le m_{21} \le 0.26$

Following the methodology for $S_p(H)$, it is verified as being 0.73:

(1) $P(H \mid \overline{E_1} \cap \overline{E_2})$ is calculated as [0.3, 0.9], take $S_p(H \mid \overline{E_1} \cap \overline{E_2})$=0.9

(2) The conditional probabilities are ordered: C_1=0.4 ($P(H|E_2)$) < C_2=0.6 ($P(H|E_1)$) < C_3=0.9 ($P(H \mid \overline{E_1} \cap \overline{E_2})$)

(3) $M_3 = m_{22}+m_{23}+m_{32}+m_{33} = 0.43$

(4) $M_2 = m_{11}+m_{12}+m_{13}+m_{31} = 0.57-m_{21}$

(5) $M_1 = m_{21}$

(6) $S_n(H)$=max[0.4×m_{21}+0.6×(0.57-m_{21})+0.9×0.43]=0.73, at m_{21}=0 and $0.03 \le m_{12} \le 0.06$

References

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Appendix I

PERIMETA: Software guidance

This Appendix is the help documentation for the PERIMETA software that has been developed by the research group, including the author, that was engaged in the EPSRC Condition Monitoring and Asset Management for complex infrastructure systems (CMAM) project. The author was involved in designing the specification of the software and was solely responsible for the case study described in Chapter 6, whilst the software implementation was conducted by co-researchers. This documentation is included to provide a more thorough overview of the workings of the software.

I.1. Introduction

PERIMETA (Performance Through Intelligent Measurement) is a software supported methodology to manage the performance of complex infrastructure systems.

This appendix explains the features of the software, how the tool is used to create and analyse system performance and an explanation of the key terminology. The appendix is organised into the following sections:

- An overview of the software tool and the main GUI elements.
- Creating processes and links in the model.
- Adding performance indicators.
- Viewing different aspects of performance.
- Calculating evidence propogation.
- A glossary of terms.

I.2. Overview

After launching the PERIMETA application a PERIMETA model needs to be created. Either a new empty model is created or a previously saved model may be opened from a PERIMETA document. See File menu options for more details. The PERIMETA model encapsulates the process model and links, the performance indicators and the aspects of performance. Currently only one PERIMETA document at a time may be opened. The user interface is split into three main areas, indicated in Figure I.1.

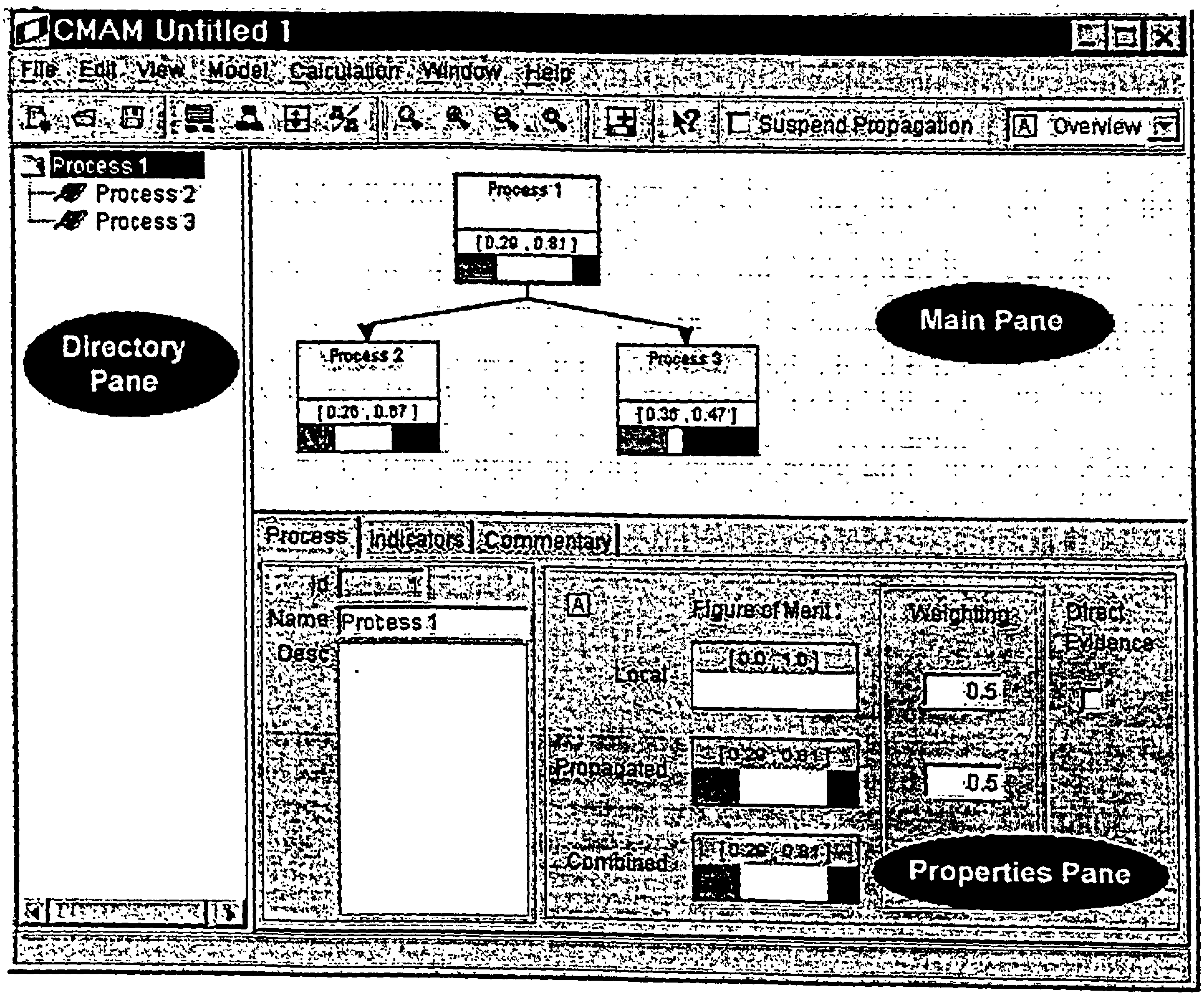

Figure I.1 The main pane, properties pane and directory pane of the PERIMETA software

The *main pane* displays either the graphical network view of the processes or the performance indicator search results. To switch between these use menu options View | Process Graph or View | Performance Indicators, or the related toolbar buttons.

The *properties pane* displays details about the currently selected object in the main pane. Selecting the background in the graphical view will display summary details for the model. Selecting more than one object will display nothing.

The *directory pane* displays the graphical network of processes as a hierarchical tree view. Selecting processes in the directory pane will display the processes in the main pane.

I.2.1. Menu

The main window shows seven drop-down menu options:

<u>File</u> <u>Edit</u> <u>View</u> <u>Model</u> <u>Calculation</u> <u>Window</u> <u>Help</u>

File Menu

New	Create a new empty PERIMETA model with no processes or performance indicators. The model is given a default name.
Open	Open an existing PERIMETA model from a PERIMETA document. These are stored as XML files and have an .xml extension. The document must match the PERIMETA XML schema, cmam-model.xsd, or it can not be openned.
Close	Close the currently displayed PERIMETA model without ending the application.
Save	Save the PERIMETA model to a PERIMETA document. If previously saved then will be saved to the same file, otherwise a save dialog will prompt for a file name.
Save As	Save the PERIMETA model to a PERIMETA document and specify the save file name.
Save As JPEG	Save the process graph image as a JPEG file.
Exit	Exit the application.

Edit Menu

Undo	Undo last edit.
Copy	Copy selection to the clipboard. Use to copy processes and links in the process graph view or performance indicators in the performance indicator view. Note that copied processes also include copies of associated performance indicators, however copied performance indicators do not include copies of associated processes.
Paste	Paste the selection from the clipboard.
Delete	Delete the current selection(s) in the graph or performance indicator views.
Select All	Select all processes and links in the process graph view.

View Menu

Zoom In	Zoom into the process graph view.
Zoom Normal	Reset to the normal scale in the process graph view.
Zoom Out	Zoom out from the process graph view.
Zoom To Fit	Zoom to a scale such that the entire process graph fits into the current main pane.
Process Graph	Switch to the process graph view.
Performance Indicators	Switch to the performance indicator view.
Split Process Graph	Split (or reform) the process graph into two views of the same model. The top view is read only, the bottom allows changes.
Show/Hide Grid	Controls whether the grid in the graph view is visible.
Show/Hide Link Labels	Display the necessity/sufficiency as a label over the link in the graph view.

Model Menu

Add Process	Create a new process with default attributes.
Add Performance Indicator	Create a new performance indicator with default attributes.
Import Performance Indicators	Performance indicators created in another application may be imported from an XML file. The document must match the PERIMETA performance indicator schema, cmam-perfind.xsd.
Maintain Performance Aspect	Add, delete or update performance aspects.

Calculation Menu

Suspend Evidence Propagation	Disable/Enable changes in local evidence from being propagated up through the network.
Recalculate Evidence Propagation	Recalculate propagated evidence for all process nodes in the network for all aspects of performance.
Calculation Preferences	Allow the learning rate and necessity power to be set for this PERIMETA model. These variables are used to approximate the 2^*N* judgements to 2*N* sufficiency and necessity judgements for *N* processes. Also enables the direct input of conditional probabilities (see calculation preferences for more information).

Window Menu

Directory Pane	Show/Hide the directory pane.
Properties Pane	Show/Hide the properties pane.

Help Menu

Help Topics	
Field-Level Help	Go straight to the relevant help section appropriate to the GUI control in the application. Only some controls are set-up for this, if not the top level help is displayed.
About PERIMETA	Information about PERIMETA

I.2.2. Toolbar

The controls in the toolbar are listed below. Follow the link to the menu options for a fuller description of the action.

Icon	Menu equivalent	Description
	File | New	Create a new empty PERIMETA model.
	File | Open	Open an existing PERIMETA document.
	File | Save	Save the PERIMETA model to a PERIMETA document.
	View | Performance Indicators	Switch to the performance indicator view.
	View | Process Graph	Switch to the process graph view.
	Window | Properties Pane	Show or hide the properties pane.
	View | Split Process Graph	Split or reform the process graph view.
	View | Zoom Normal	Show process graph view at normal size.
	View | Zoom In	Zoom in to process graph.
	View | Zoom Out	Zoom out of process graph.
	View | Zoom To Fit	Zoom to show entire process graph in main pane.
	Model | Add Process	Add a new process to the graph.
	Help | Field-Level Help	Enter field level help mode.
	Calculation | Suspend Evidence Propagation	Prevent evidence from being automatically propagated.
Overview		Choose the current aspect of performance view.

I.3. CREATING A NEW PROCESS MODEL

I.3.1. Introduction

A PERIMETA model graph defines the parent and child processes which form the engineering system under consideration. The graph is formally a Directed Acyclic Graph (DAG).

Directed: It has direction from top to bottom as super processes are broken down into sub-processes.
Acyclic: Cycles of child to parent back to child are not allowed (an error message is displayed in the status bar if such a link is attempted).
Graph: A child process may feed into more than one parent.

Evidence is applied to the PERIMETA model by assigning performance indicators to a process or by entering local evidence (Figure of Merit) directly. This evidence is propagated through the model using interval probability theory. Although the direction of the graph is top down for process decomposition, the propagation of evidence is bottom up.

A tree view of the graph is presented in the directory pane. This allows larger graphs to be inspected and navigated in a more compact form. Note that converting a DAG to a tree representation requires that child processes with multiple parents appear more than once.

I.3.2. Presenting the evidence

The evidence for and against a proposition represented by a process is displayed with an 'Italian Flag' symbol where green is evidence for, red is evidence against and white is the uncertainty.

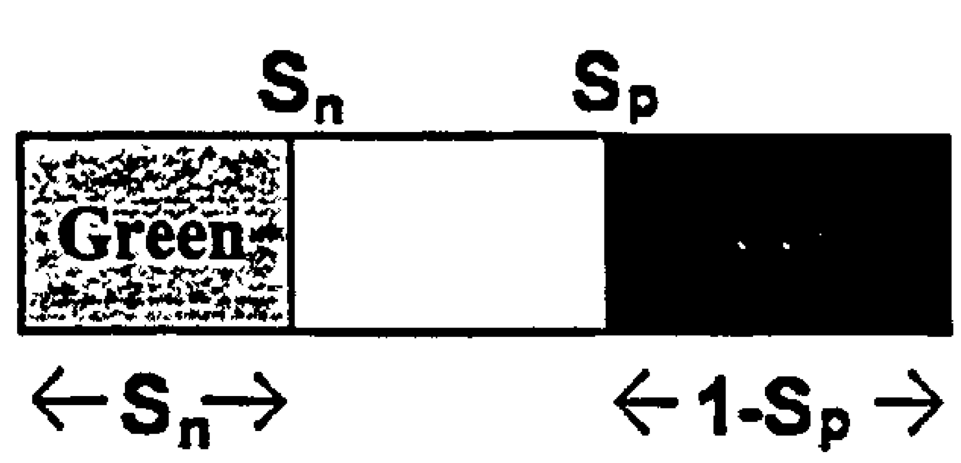

Figure I.2 The Italian Flag showing evidence for and against a proposition

The evidence for is *Sn* and the evidence against is 1-*Sp*. The interval pair numeric value is displayed as [*Sn*, *Sp*].

I.3.3. Creating the model

Use menu option Model | Add Process or the toolbar button to add a new process.
Press mouse down and drag to change the process position.
Place the mouse over the top or bottom of the process to highlight the link end points (blue oval).
Drag to create a link between processes, the link will snap to the nearest valid end point.
Highlighting a process or a link by clicking on it will load the property pane with the process attributes or link attributes. Use the property pane to edit details about the process or link.

I.3.4. Process Attributes

The process attributes are split between three tabs. All user entered values are saved to the PERIMETA model in memory by pressing enter or by tabbing off the control. A process has a number of attributes that can be associated with it. These are listed in the following sections.

Description and evidence

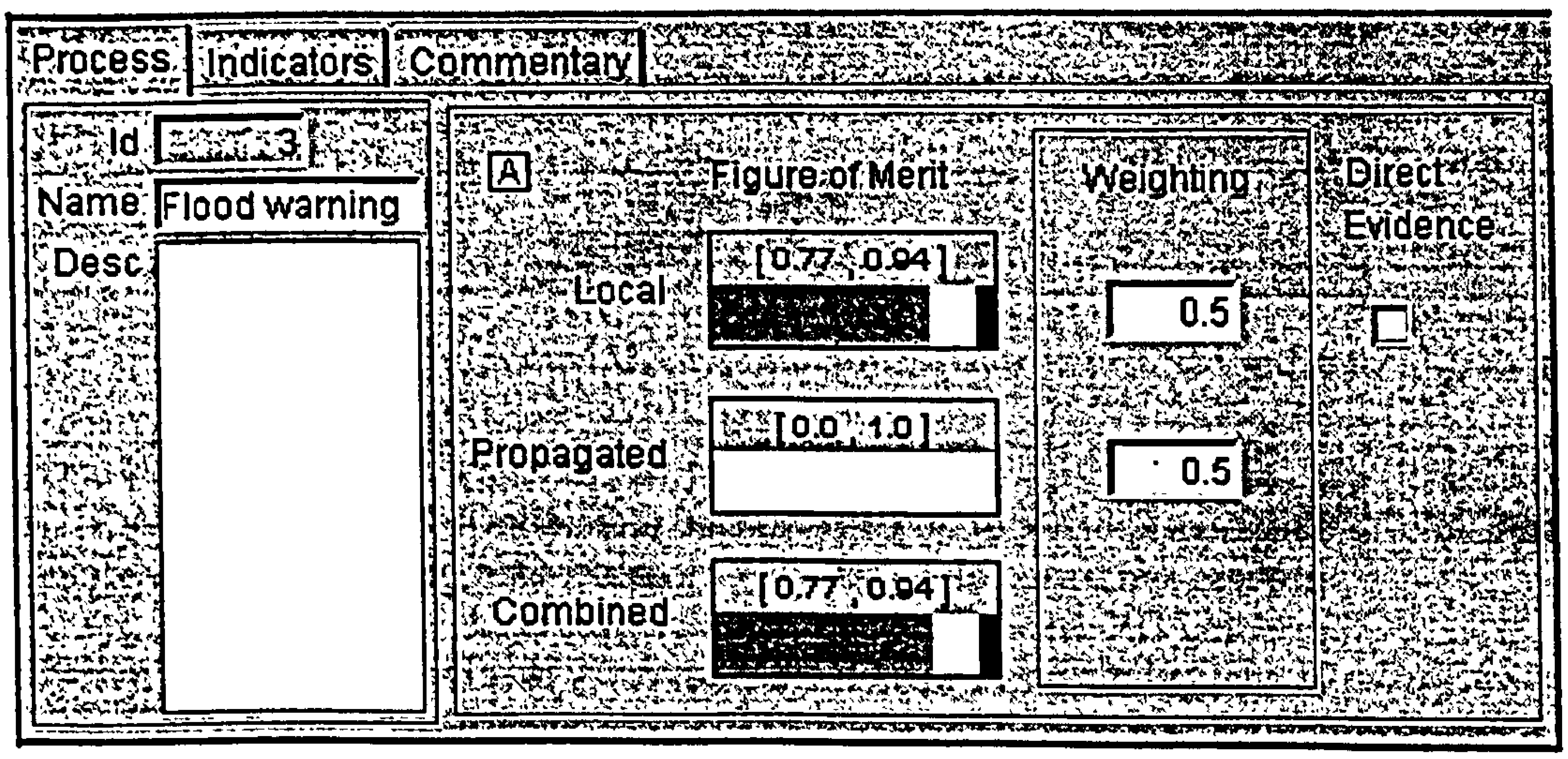

Figure I.3 Pane showing process properties

ID	Is a unique identifier for the process within the model and is assigned automatically.
Name	Is a user supplied name.
Desc	Is an optional description of the process.
Local Figure of Merit	Is either read only and calculated from assigned performance indicators or may be assigned by ticking 'Direct Evidence' and dragging from either end of the bar. If there is no local FOM this is displayed as the interval [0,1].
Propagated Figure of Merit	Is read only and displays evidence contributed from children of this process. If there is no local FOM this is displayed as the interval [0,1].
Combined Figure of Merit	Is read only and is a weighted sum of local and propagated FOM. If there is no local FOM the weights are ignored and the propagated FOM is used (and vice versa).
Local Weighting	Weights the contribution of local FOM.
Propagated Weighting	Weights the contribution of propagated FOM.
Direct Evidence	Allows local FOM to be set directly for this process by clicking and dragging on the local FOM italian flag.

Associate performance indicators

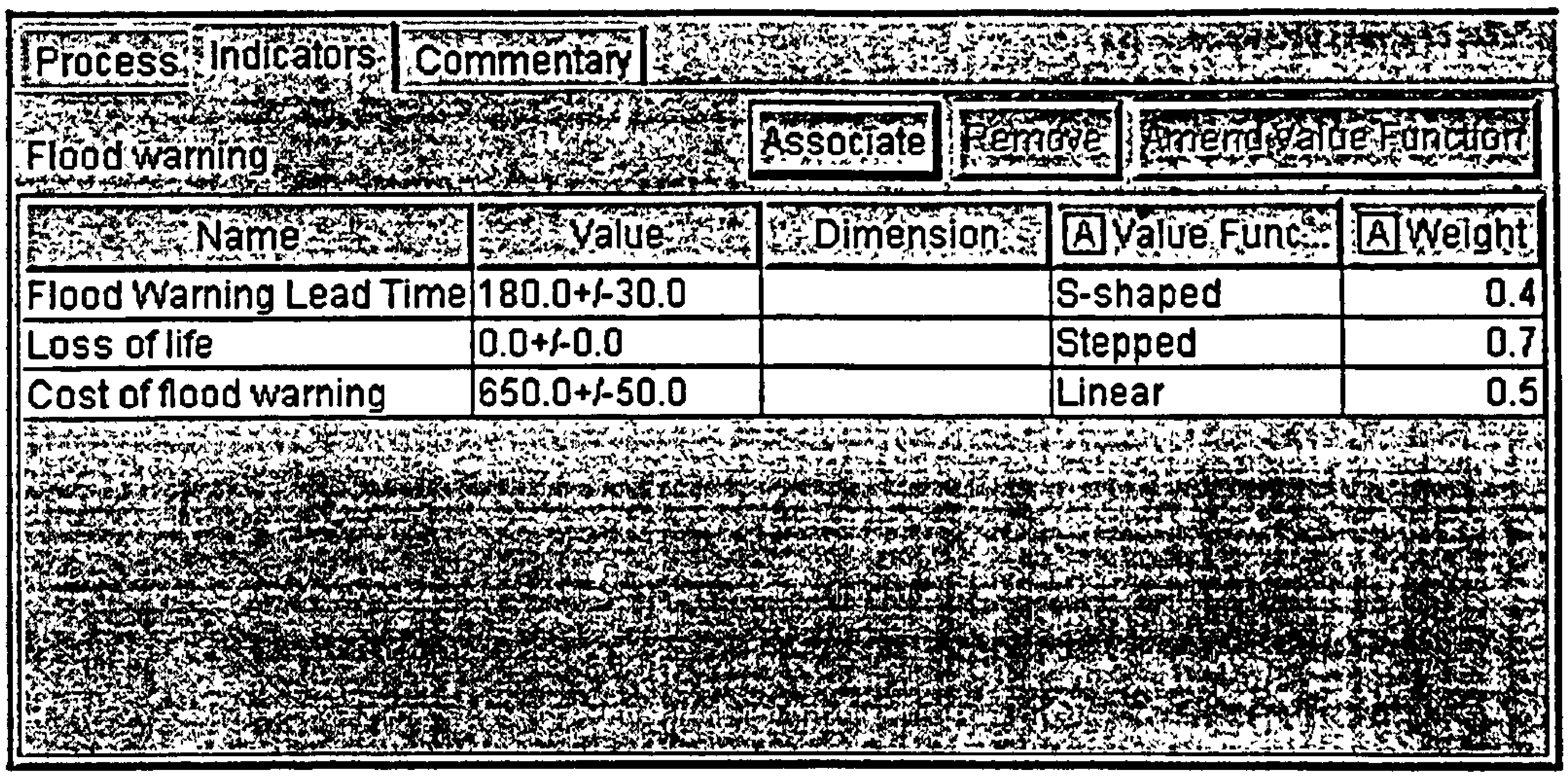

Figure I.4 Pane showing performance indicators associated with a process

Associate	Displays a dialog of all performance indicators not already assigned to this process and sorted alphabetically by name. One or more may be selected and pressing OK associates them with this process.
Remove	Is enabled if a performance indicator entry is selected, it then removes the association with the process.
Amend Value Function	Allows an aspect specific value function to be assigned to this process/performance indicator combination.
Weight	Is used to combine performance indicators. A value between 0 and 1 may be entered and this is normalised when applied to calculate the local FOM from the non dimensional performance indicator values.

Commentary

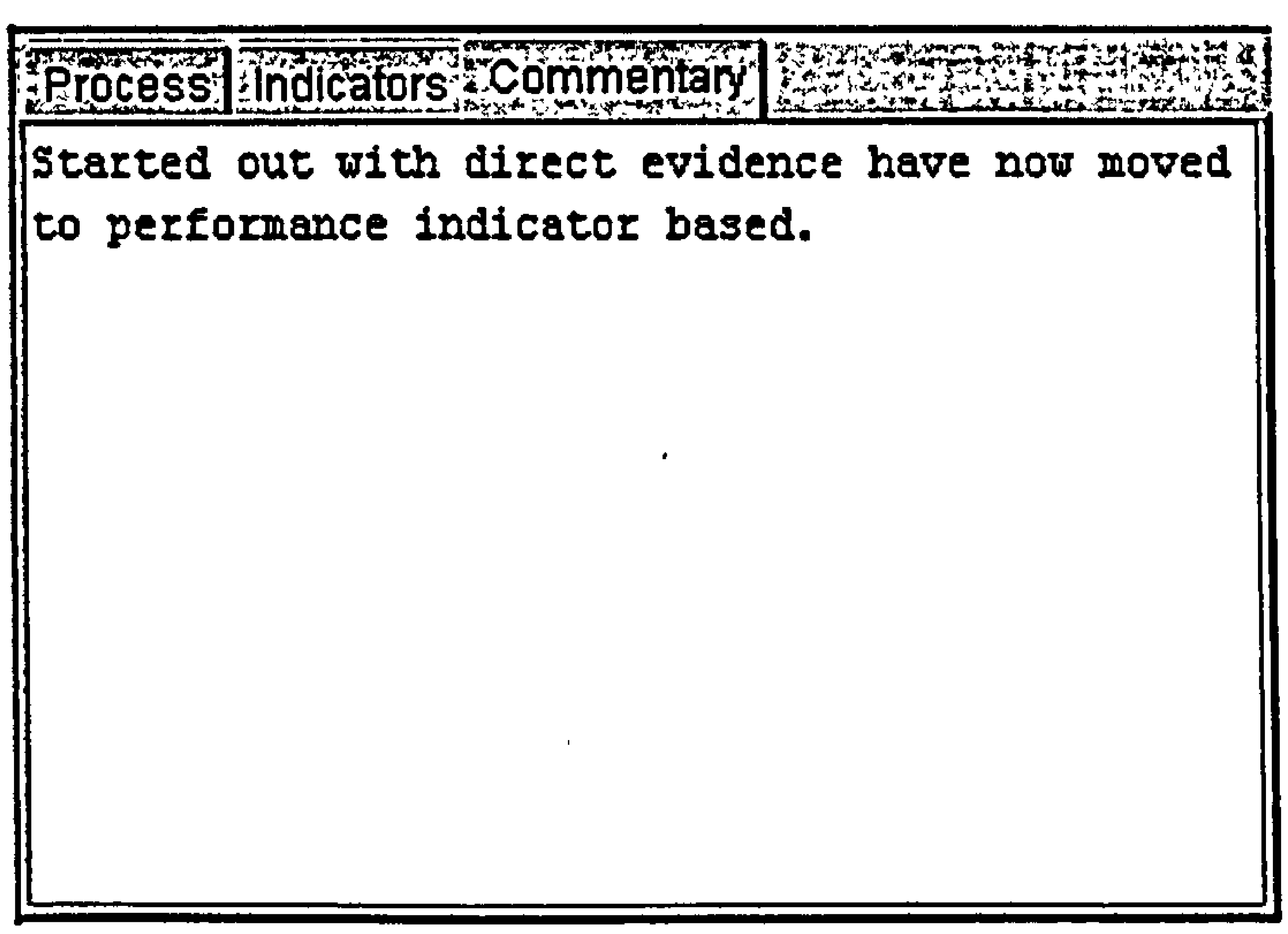

Figure I.5 Process specific commentary box

Commentary Allows supporting information to be recorded.

I.3.5. Value Functions

The value functions are predefined within the PERIMETA software and consist of one linguistic and five numeric value functions.

Linguistic

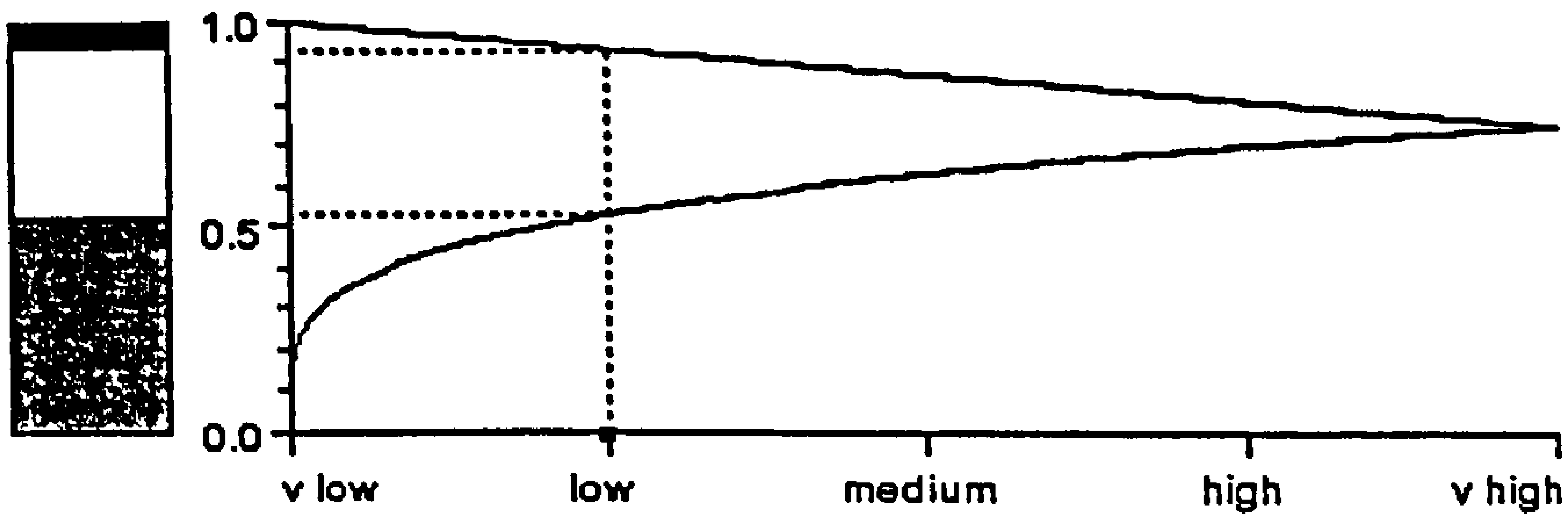

Figure I.6 Linguistic Performance Indicator

The linguistic value function comprises a set of curves, one for each of the five performance values (very poor, poor, medium, good, very good). The curve above is for good performance. The x-axis scale takes five discrete values representing confidence in the performance value.

Numeric

Each numeric value function requires an upper and lower bound to be set. By reversing these, *i.e.* making the upper bound less than the lower bound, the sign of the curve is reversed. The bounds are used to limit the acceptable measured values.

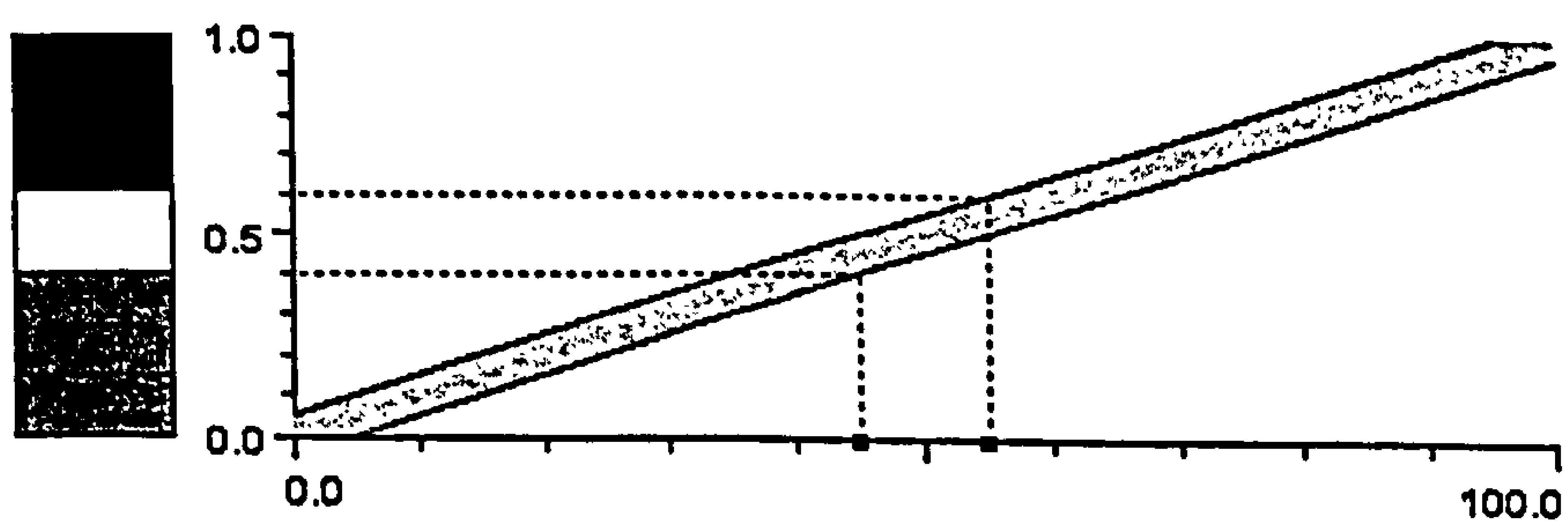

Figure I.7 Linear value function.

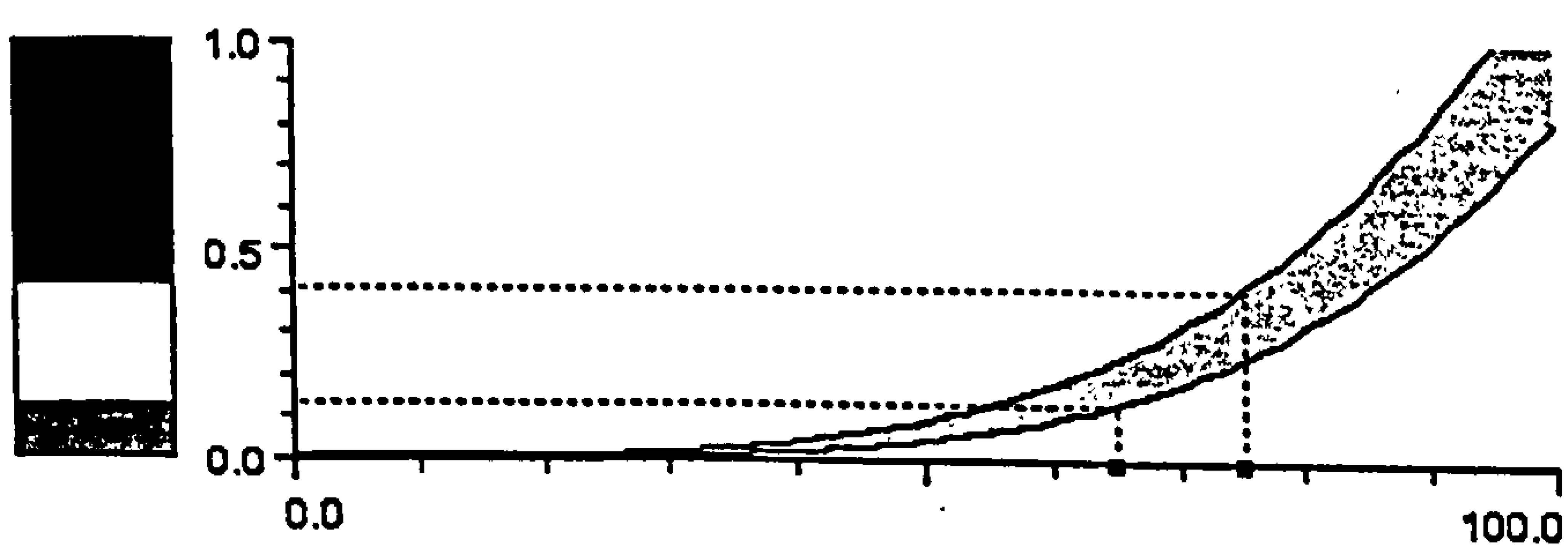

Figure I.8 Concave value function.

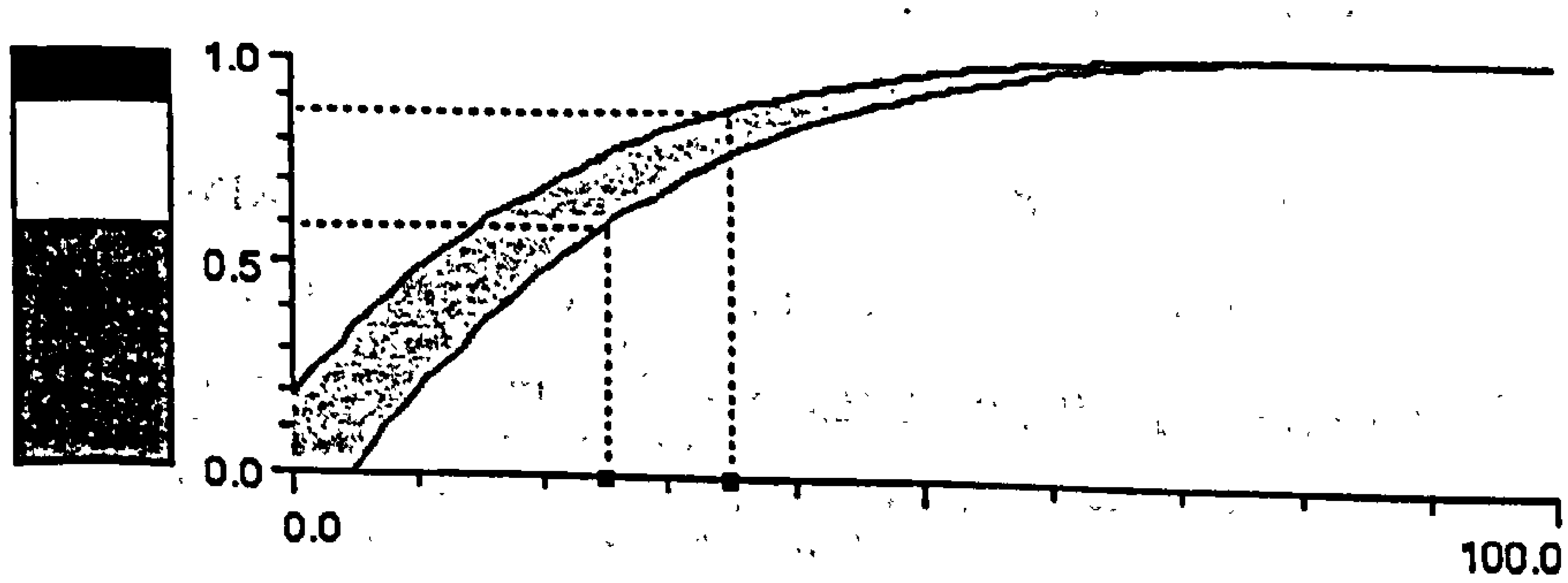

Figure I.9 Convex value function.

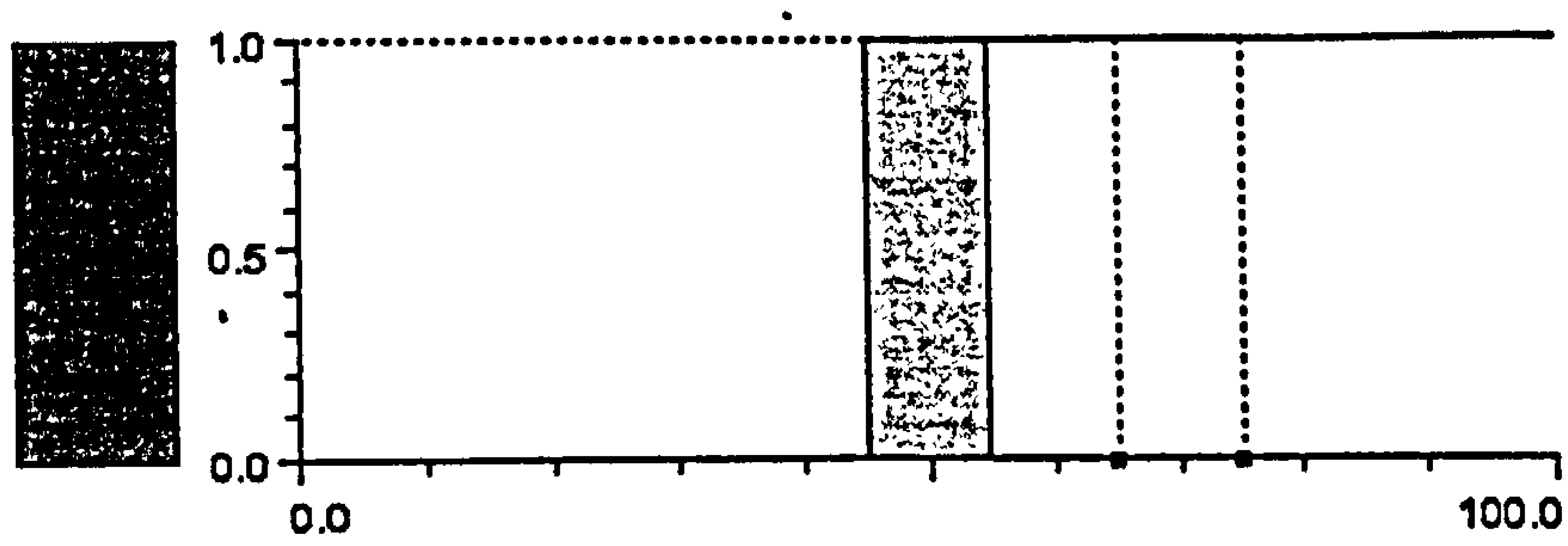

Figure I.10 Stepped value function.

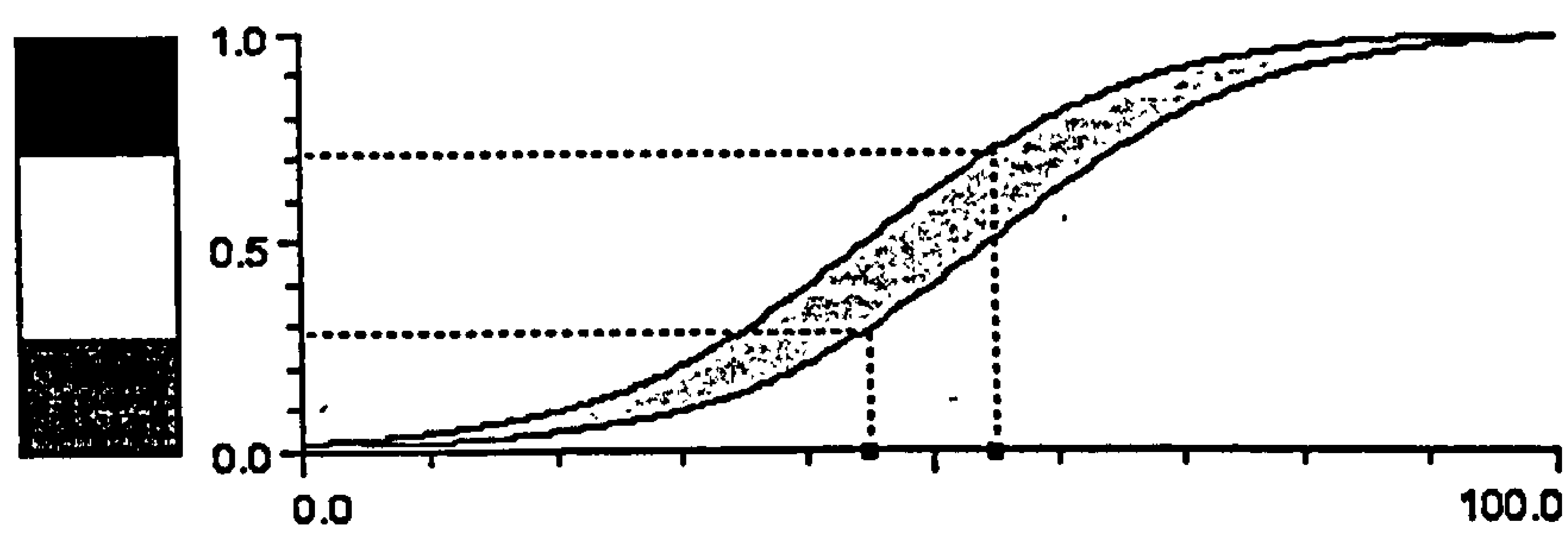

Figure I.11 S-shaped value function.

I.3.6. Performance Indicators

Introduction

Performance indicators represent evidence about the performance of a process. This evidence is assembled from all available sources, including monitoring measurements, inspection records, design calculations and expert judgements. The measured value of performance may be input as a numeric value or a linguistic statement.

Measured performance will typically be against a collection of different dimensions and so to estimate the system performance a dimensional measure must be passed through a value function to:

- map onto a common non-dimensional scale
- compare with an organisational target that represents good performance

If a process has more than one performance indicator these may be weighted according to relative importance (see process attributes).

Performance indicators are created and exist separately from the PERIMETA model graph of processes. They are not dependent on the aspect of performance, although when assigned to processes will have aspect specific properties set. They may be assigned to more than one process including processes in the same path, for example a parent and child process.

Adding/Deleting

Performance indicators are stored as a list within the PERIMETA model. They are not displayed by default, to show some or all of the list use the search button. The results are displayed in a table:

Performance Indicator Search Results

Name	Value	Dimension	Value Function	Processes
3161.02.10.L08 - Overturning	1.25+/-0.1		S-shaped	
3161.02.10.L08 - Sliding	1.25+/-0.0		S-shaped	
3161.02.10.L09 - Overturning	1.25+/-0.0		S-shaped	
3161.02.10.L09 - Sliding	1.25+/-0.0		S-shaped	
3161.02.10.L10 - Rotn Riverw...	1.25+/-0.0		S-shaped	
Cost of flood warning	650.0+/-50.0		Linear	Flood warning
Cost of inspection strategy	2000.0+/-50.0		Linear	Sub-Reach 01
Cost of maintenance strategy	2000.0+/-50.0		Linear	Sub-Reach 01
Environmental Impacts	medium , low		Linguistic	Reach 3161.02
Flood Warning Lead Time	180.0+/-30.0		S-shaped	Flood warning

Search 150 of 150 performance indicators displayed New Delete

Figure I.12 Performance indicator search result pane

The table is a read only list of the important attributes sorted alphabetically by name. The total number of performance indicators displayed and the total number available in the model are displayed at the bottom of the dialog. The processes column lists only the first process associated with the performance indicator. If more than one is present the process is followed by dots.

To add a new performance indicator with default attributes press the **New** button. To delete an existing performance indicator select the table entry and press the **Delete** button.

Highlighting a performance indicator in the results table by clicking on it will load the property pane with the performance indicator attributes. Use the property pane to edit the details.

A performance indicator may be associated with a process by dragging and dropping from the results table to the process in the directory view.

Performance indicators may be imported into a PERIMETA model from an external source by creating an xml document which meets the cmam-perfind.xsd xml schema. Run Model | Import Performance Indicators to load the file. Any performance indicators supplied without an ID are added as new, those with an ID are used to update an existing indicator. Any errors are reported to the screen after running the import.

I.3.7. Performance Indicator Attributes

The performance indicator attributes are split between three tabs (the two greyed out tabs have not been implemented). Most user entered values are saved to the PERIMETA model in memory by pressing enter or by tabbing off the control, however because of the relationship between attributes some must be explicitly saved using the ☑ icon.

Name and Value

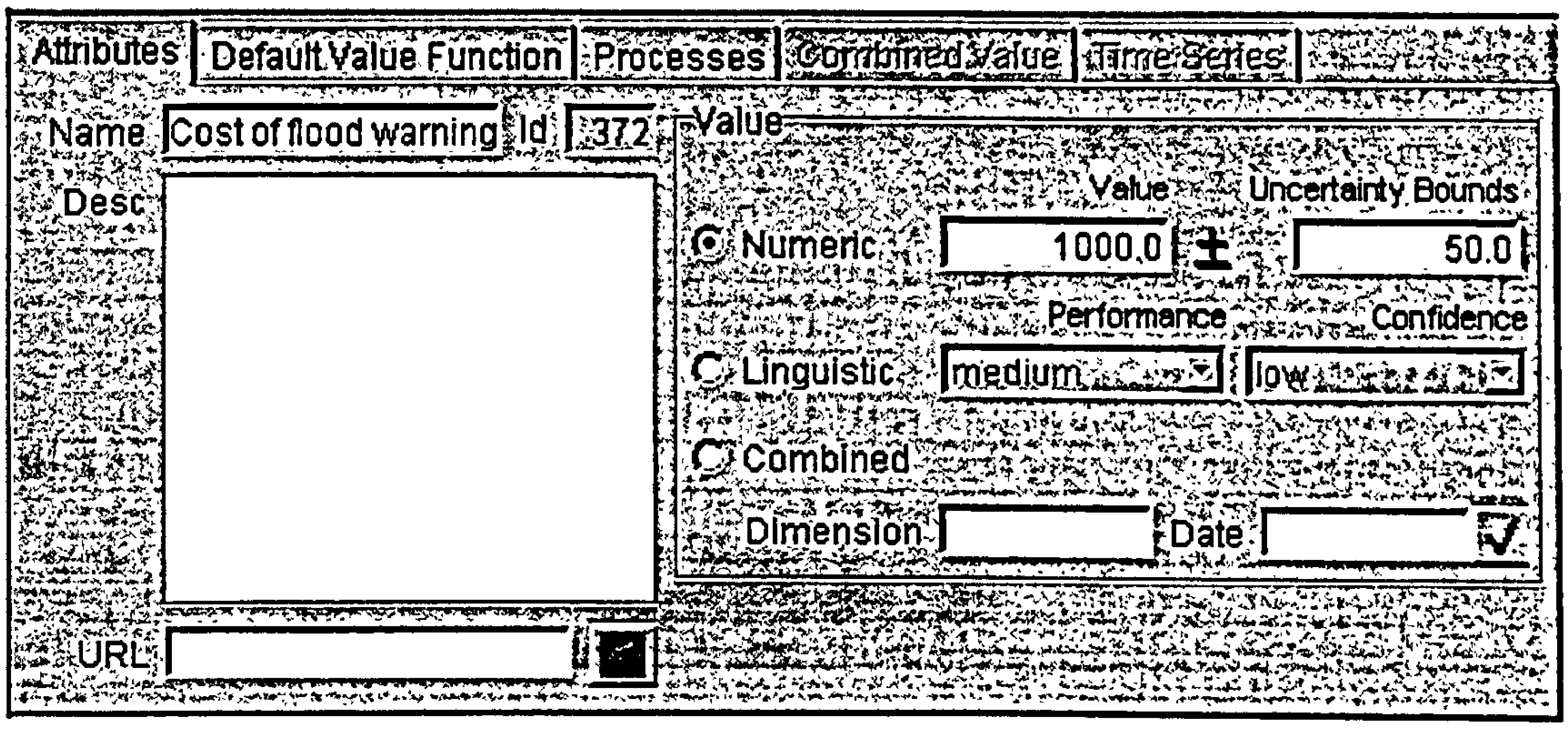

Figure I.13 Pane showing attributes of a 'Performance Indicator'

Name	Is a user supplied name.
ID	Is a unique identifier for the performance indicator within the model and is assigned automatically.
Desc	Is an optional description of the performance indicator.
URL	Is an optional identifier for the source of the information.
Numeric Value	Is the measured value.
Uncertainty Bounds	Is an optional numeric uncertainty on the measured value.
Linguistic Performance	Is a linguistic judgement of system performance on a 5 point scale of 'very poor' to 'very good'.
Linguistic Confidence	Is a linguistic judgement of confidence in the measure on a 5 point scale of 'very low' to 'very high'.
Combined	Is a feature not yet implemented to indicate that the performance indicator is a combination of other performance indicators.
Dimension	Is an optional description of the measurement dimension.
Date	Is an optional measurement date which allows a time series plot (not yet implemented) to be produced.

When the value attributes are saved by pressing the red tick icon a check is made that the value type (numeric or linguistic) matches the default value function. A warning is issued and the value function is changed to the numeric default (linear) or to the linguistic value function.

Default Value Function

The default value function is used when a performance indicator is assigned to a process unless an aspect specific value function is set.

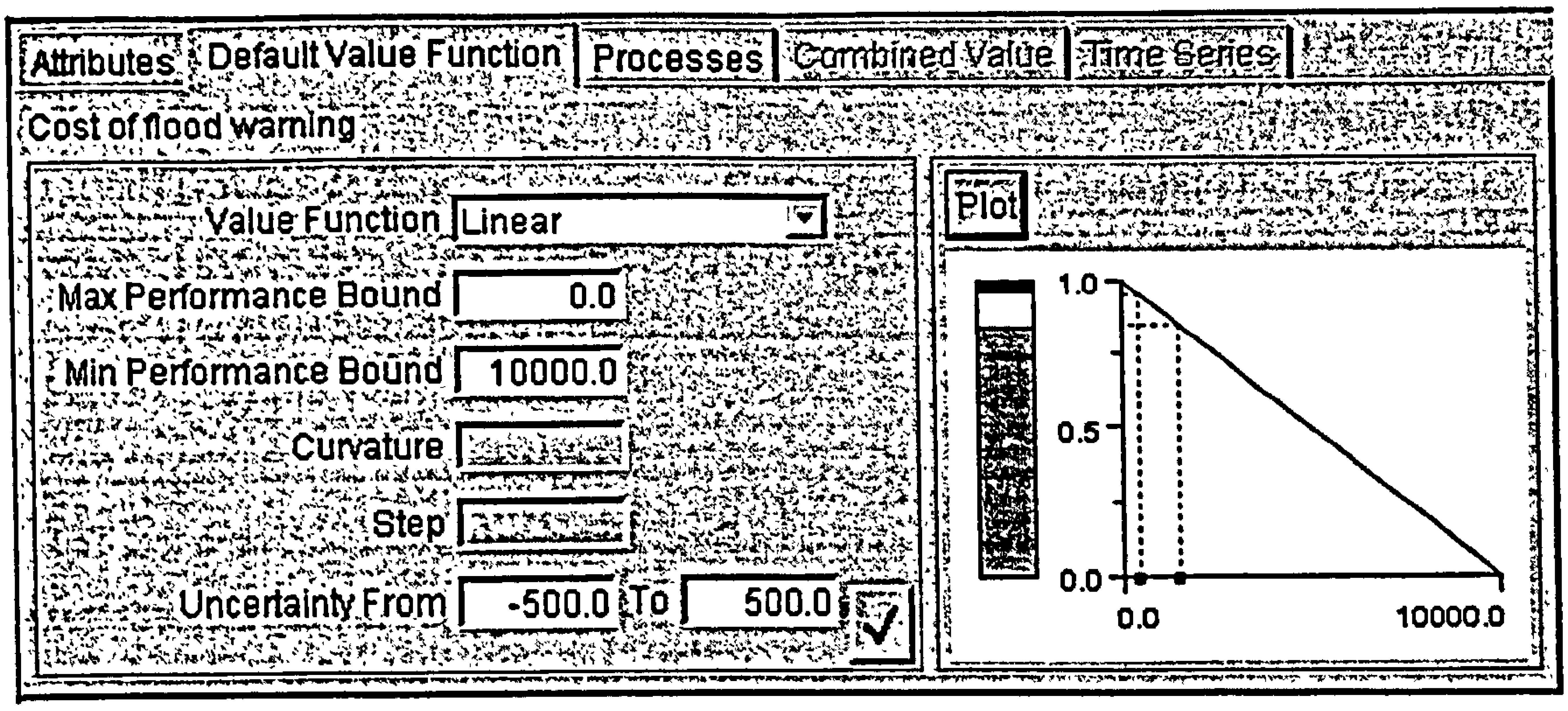

Figure I.14 The default value function pane

Value Function Allows one of the pre-defined value functions to be selected. Each value function has one or more properties to be set from the following list.

Max Performance Bound Sets the measured value at which performance is 1. A value outside this bound will be set to a performance of 1.

Min Performance Bound Sets the measured value at which performance is 0. A value outside this bound will be set to a performance of 0.

Curvature Sets the amount of curvature on convex, concave and s-shaped curves.

Step Sets the step position on stepped and s-shaped curves.

Uncertainty From Is an optional uncertainty for the value function and must be less than or equal to zero.

Uncertainty To Is an optional uncertainty for the value function and must be greater than or equal to zero.

Plot Shows the value function graph plotted as measured value (x-axis) against non dimensional performance (y-axis).

Processes

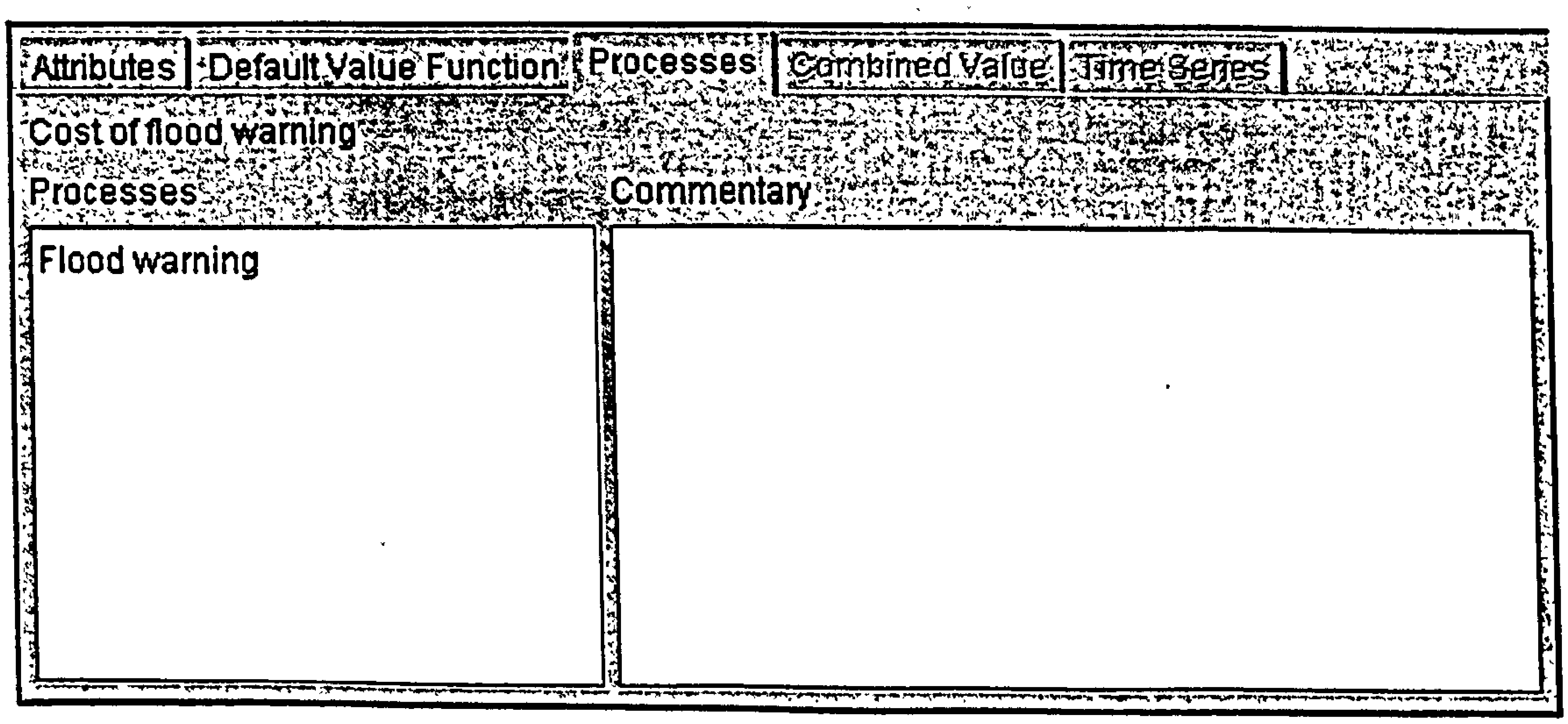

Figure I.15 Process commentary pane

Processes Displays a read only list of process nodes associated with this performance indicator.

Commentary Provides a text area to record general notes about the performance indicator.

I.4. ASPECTS OF PERFORMANCE

I.4.1. Introduction

Figure of merit gives a summary of system performance against a range of objectives. This is the overview aspect of performance that all PERIMETA models must have. Performance may also be measured against different objectives such as cost, safety or environment and these give rise to alternative aspects of performance.

This is achieved by altering certain system attributes for different aspects of performance. The weighting of performance indicators is one such attribute and can be used to increase or decrease the weight of a measure according to the aspect of performance. A list of attributes dependent on aspect are detailed below.

The PERIMETA software allows an aspect of performance to be selected with a drop down list found on the toolbar, so that performance for that aspect can be displayed. Visual clues are provided, using an aspect colour, to remind the user of the current aspect. The process graph view in the main pane uses the aspect colour as a background colour and an aspect icon A is displayed where ever aspect related data is displayed.

I.4.2. Maintaining aspects

Aspects of performance are added, updated or deleted with the menu option Model | Maintain Performance Aspect.

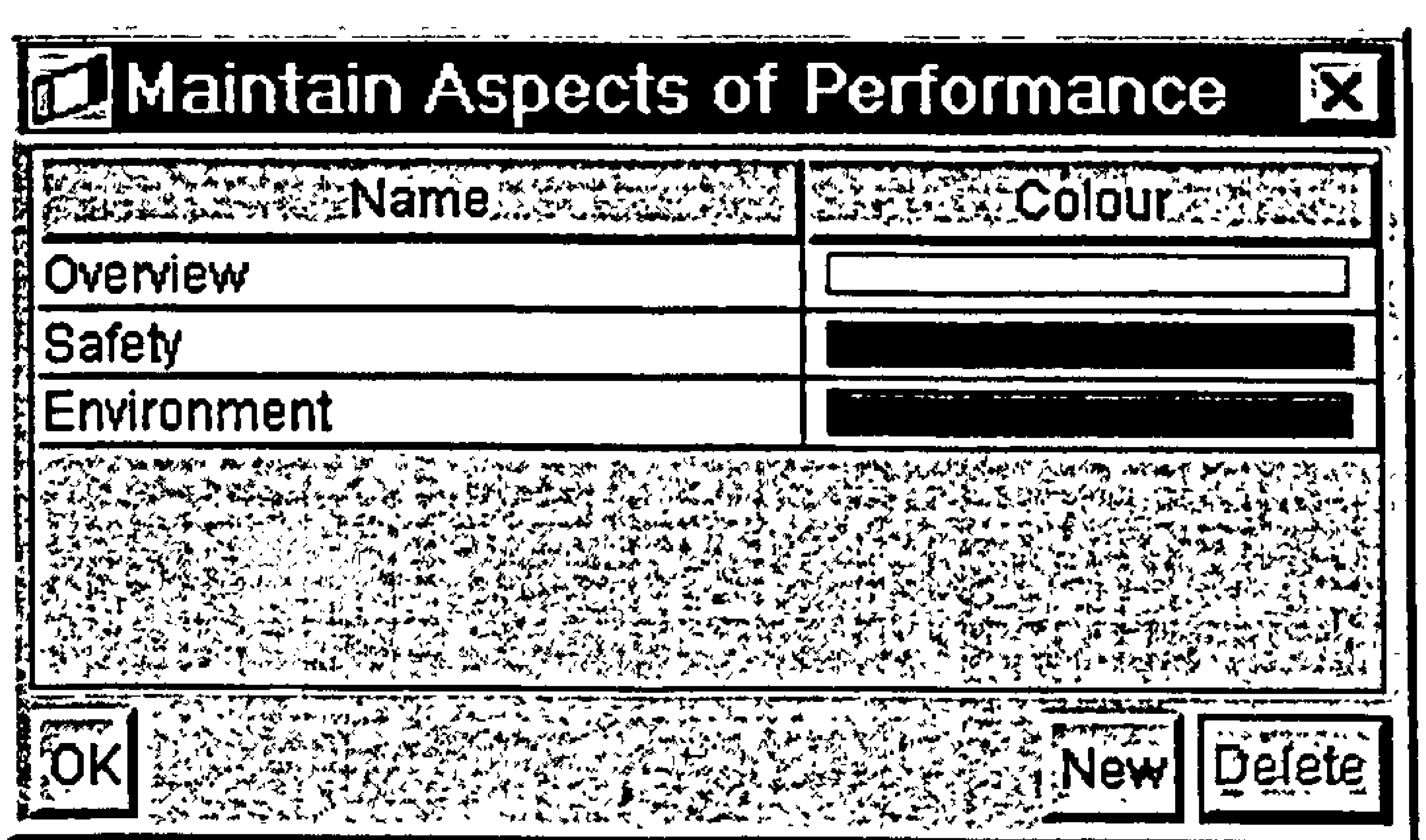

Figure I.16 Maintaining aspects of performance pane

New creates a new aspect of performance. Click on the name to edit the aspect name and click on the colour to launch a colour chooser dialog and set the aspect colour. Highlight an aspect and click on **Delete** to delete an aspect. This will raise a warning as all the attributes relating to the aspect within the model will be deleted. The overview aspect cannot be deleted.

When a new aspect is created the system attributes from the overview are copied to the new aspect. If any aspect related attributes are changed for either the new aspect or the overview the two aspects will no longer be the same.

I.4.3. Aspect related attributes

The attributes of each process and link within a PERIMETA model which are dependent upon aspect of performance are shown below . Note that processes, the process graph and performance indicators are independent of aspect of performance. So for example adding a process or performance indicator will add it to all aspects of performance.

Process	Figure of Merit	Local
		Propagated
		Combined
	Weighting	Local
		Propagated
	Direct Evidence Flag	
	Performance Indicator	Value Function
		Value Function Weight

Link	Necessity	
	Sufficiency	
	All Children Dependency	
	Pair-wise Dependency	
	Conditional Probabilities	

I.4.4. Aspect specific value function

When a performance indicator is assigned to a process there is the option of overriding the default value function supplied by the performance indicator. This is specific to the current aspect of performance displayed and so allows a different interpretation of performance measures according to aspect. For example a measure of increasing overtime on operational maintenance may show as better performance for an operational view, but as worse performance for a financial view.

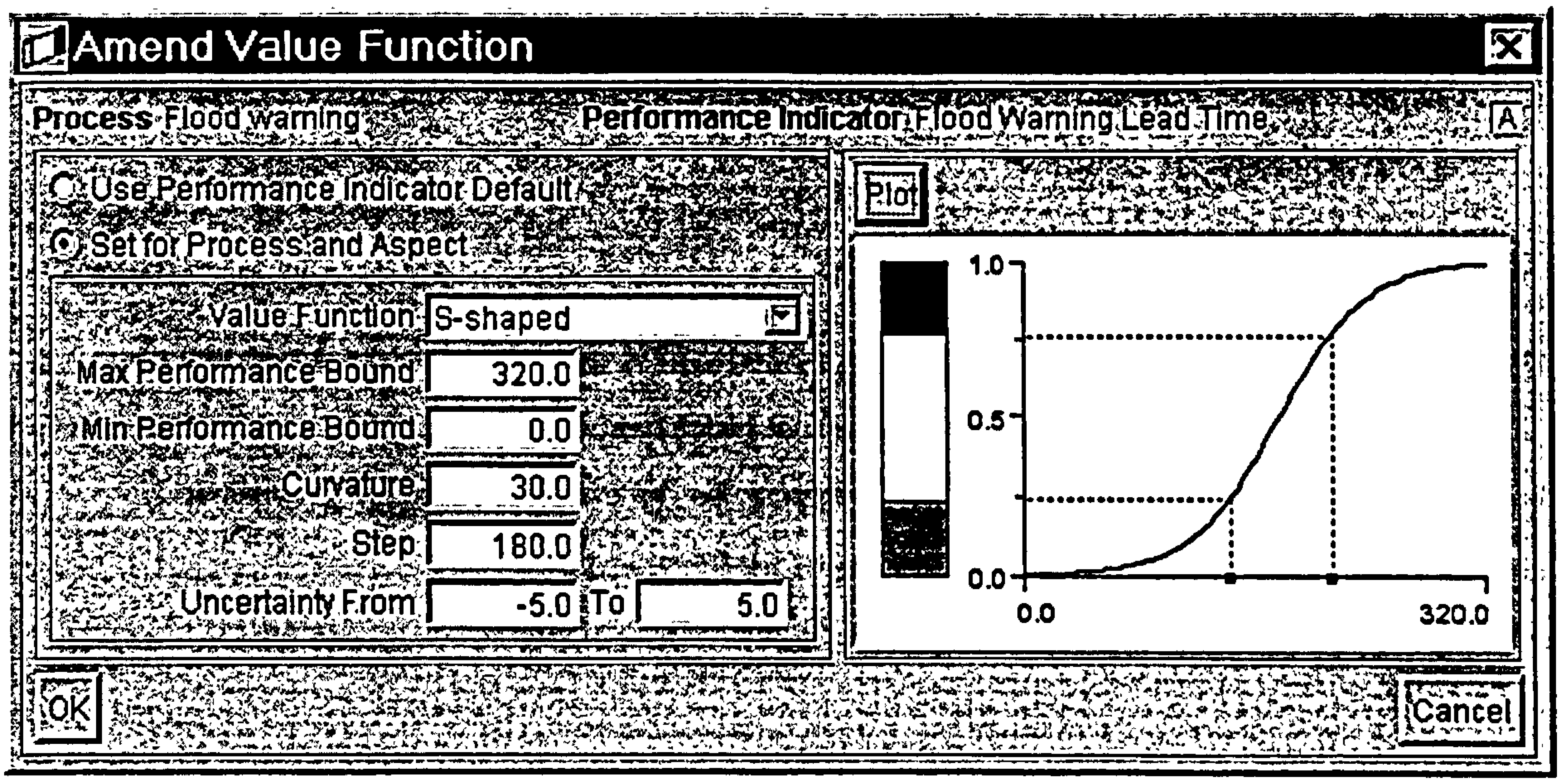

Figure I.17 Pane for amending a value function

Use Performance Indicator Default	Sets this process - performance indicator to use the default value function.
Set for Process and Aspect	Create a value function for this process - performance indicator and aspect.
Value function attributes	As for the default value function.

I.5. Calculating Evidence Propagation

I.5.1. Introduction

Evidence is calculated locally for a process to give the local figure of merit but it is also dependent on the performance of sub-systems. Evidence is propagated from the child processes to give a propagated figure of merit. Local and propagated are combined to give a weighted sum known as the combined figure of merit.

Propagation of evidence is achieved using the uncertain inference mechanism of Interval Probability Theory.

Changes to system attributes which affect local figure of merit, for example changes to performance indicators or value functions, are applied immediately. Propagation of evidence through the model graph from the point of change to the top level process also happens immediately. However if the model is large and/or many changes are required (for example updating all performance indicators) the delay during propagation may be inconvenient. The menu option Calculation | Suspend Evidence Propagation and the toolbar control

stop evidence propagation from running.

After enabling propagation with the same menu option, propagation can be recalculated for all processes and all aspects of performance with menu option Calculation | Recalculate Evidence Propagation.

I.5.2. Calculation Preferences

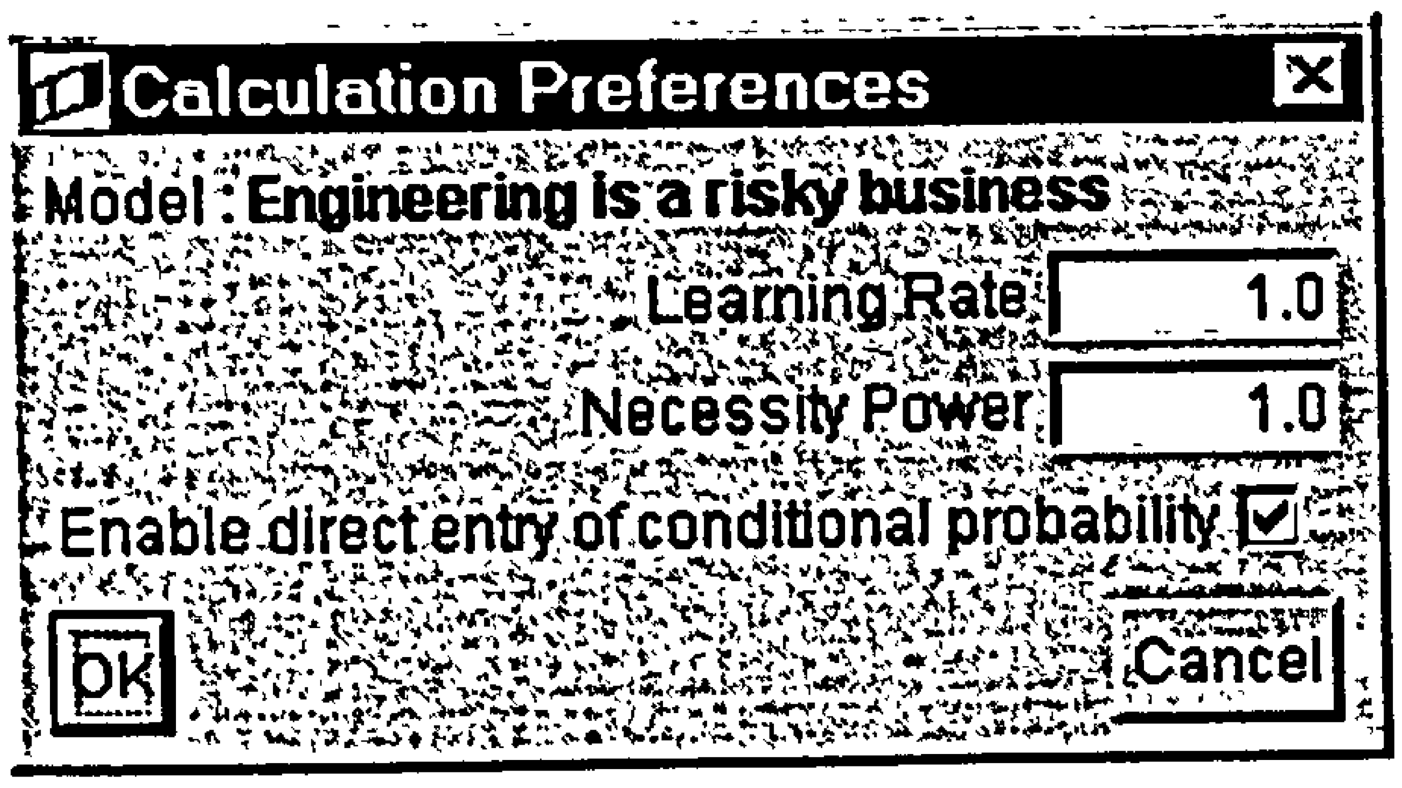

Figure I.18 Calculation preferences pane: Setting the 'Learning Rate' and 'Necessity Power'

The propagation calculation requires 2^*N* conditional probabilities to be entered for *N* child processes. However to simplify maintenance of the model only 2*N* are elicited from the user in the form of necessity and sufficiency for each link. The full number are approximated with an algorithm which requires a learning rate for sufficiency and a necessity power.

Where there are between 2 and 4 child processes the conditional probabilities may be entered directly. The 'Enable direct entry of conditional probability' check box makes the control on the link attributes tab visible.

Only the learning rate and necessity power are stored and saved with the model, the conditional probability checkbox is not saved. Default values for all fields are set for a new model.

I.5.3. Link Attributes

The link attributes are split between three tabs. The first tab is always available whilst the other two are only enabled dependent on settings for the links. The second tab, representing pairwise dependency, is only enabled if the pairwise option button is set. The third tab, representing direct entry of conditional probabilities, is only enabled if the 'set conditional probability directly' checkbox is ticked. This checkbox is only visible if enabled in calculation preferences.

All user entered values are saved to the PERIMETA model in memory by pressing enter or by tabbing off the control.

Necessity and sufficiency are displayed for the selected link. However the dependency and conditional probabilities by definition relate to the parent process of the link and so are common for all links emanating from the parent.

Necessity, Sufficiency and Shared Dependency

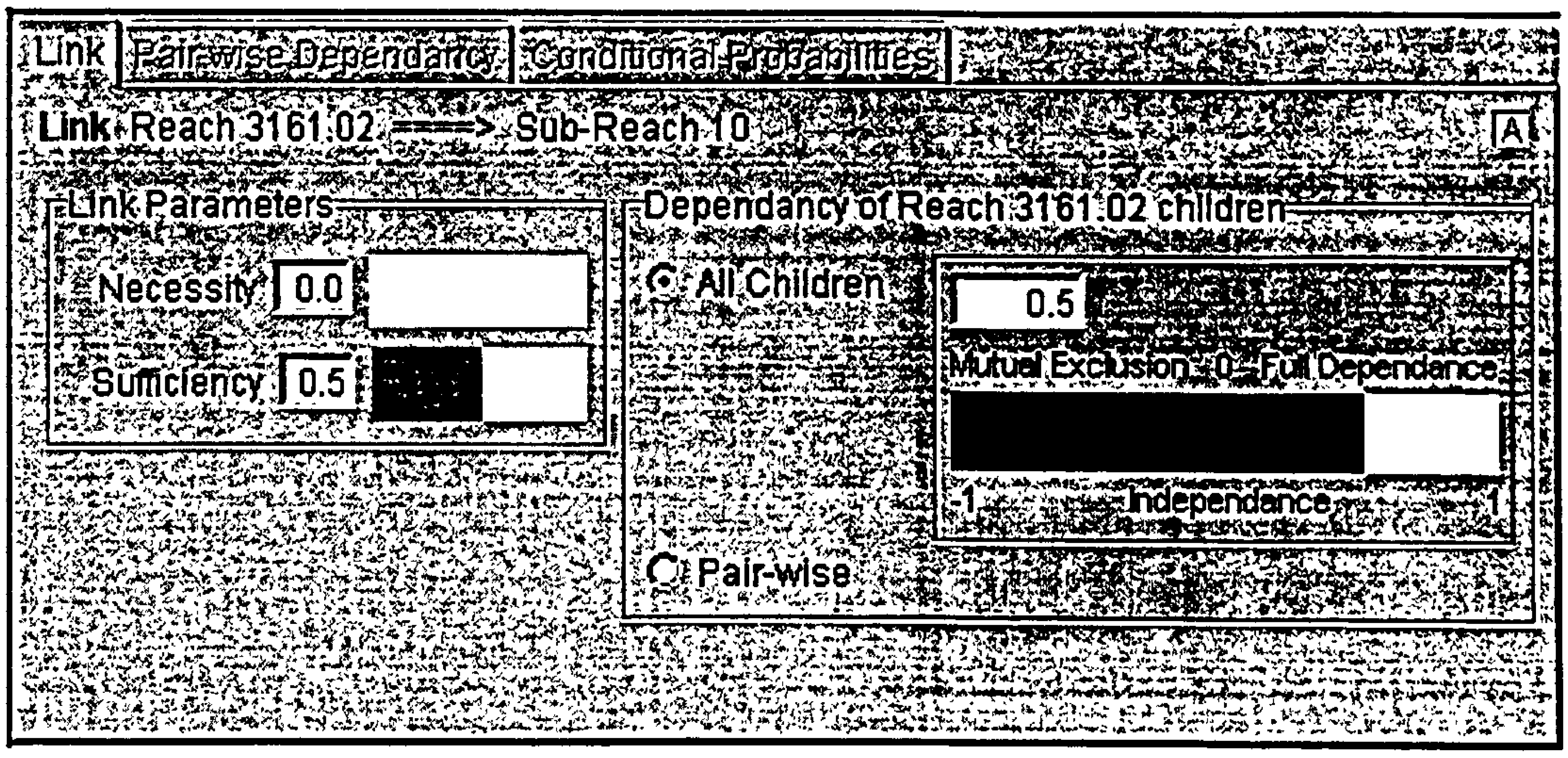

Figure I.19 Setting 'Necessity', 'Sufficiency' and 'Dependency' pane

Necessity	May be set in the interval [0,1] by typing the value or by dragging the coloured bar from the left. See Section I.5.4.
Sufficiency	May be set in the interval [0,1] by typing the value or by dragging the coloured bar from the left. See Section I.5.4.
dependency	May be set in the interval [-1,1] by typing the value or by dragging the coloured bar from the left. See Section I.5.5.
All Children	Indicates that all combinations of child pairs will be given the same dependency value.
Pair-wise	Indicates that each combination of child pairs may be given a different pair-wise dependency value. These values are entered on the Pair-wise Dependency tab.
Set conditional probability directly	This check box is only visible if direct entry of conditional probability has been enabled in the calculation preferences. Indicates that conditional probability will be entered directly and so the necessity and sufficiency controls are disabled. Conditional probability may only be entered directly for between 2 and 4 child processes.

Pairwise Dependency

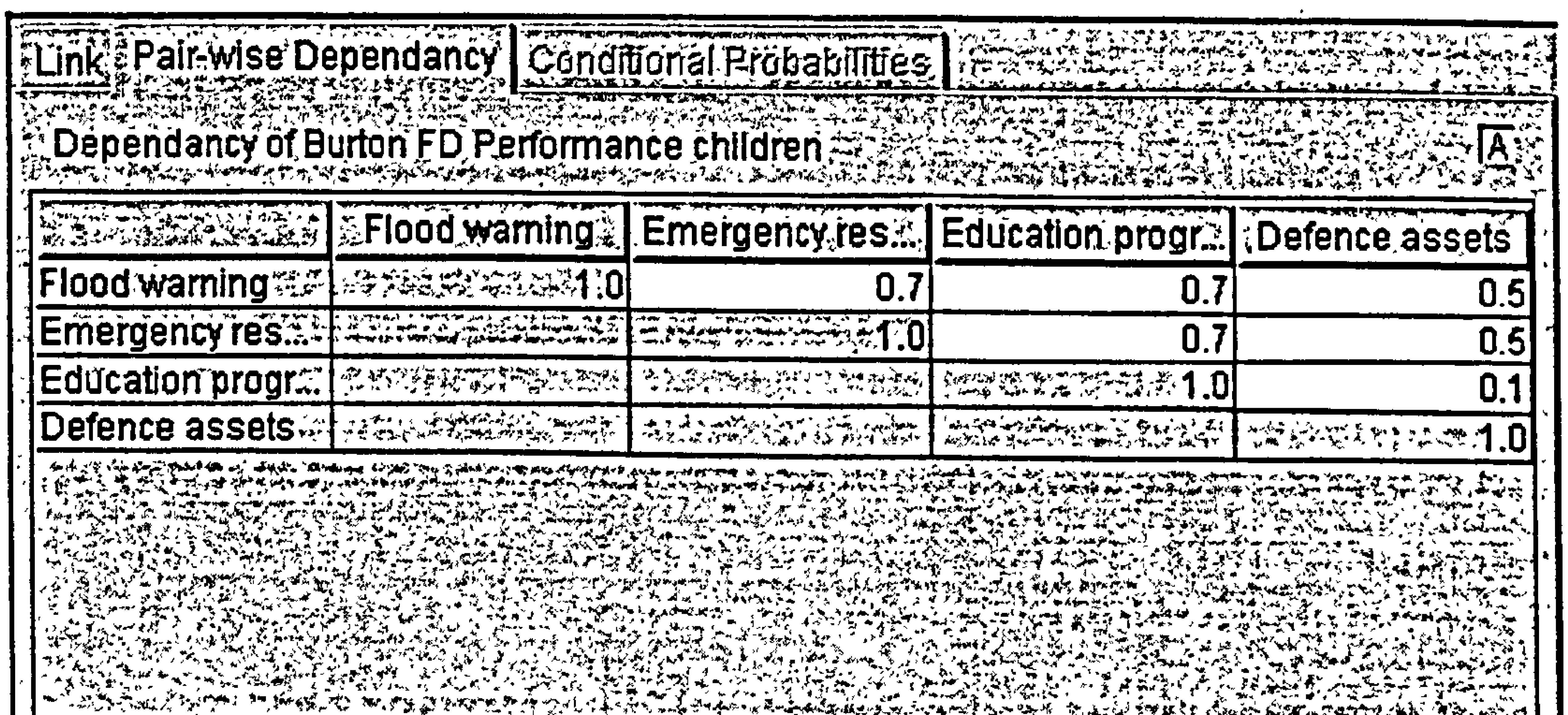

Figure I.20 Setting the 'Pairwise Dependency' pane

Dependency May be set in the interval [-1,1] by typing the value in any of the enabled table cells. Dependency of a process with itself is displayed as 1.0 but is not relevant for the propagation calculations.

Conditional Probability

Link | Pair-wise Dependancy | Conditional Probabilities

Conditional probabilities for Burton FD Performance children

E1 : Flood warning E2 : Emergency response

E3 : Education programmes E4 : Defence assets

Evidence combination	Sn Judgment	Sp Judgment
E1 ∧ E2 ∧ E3 ∧ E4	0.9634	0.9634
E1 ∧ E2 ∧ E3 ∧ ¬E4	0.4375	0.4375
E1 ∧ E2 ∧ ¬E3 ∧ E4	0.9277	0.9277
E1 ∧ E2 ∧ ¬E3 ∧ ¬E4	0.375	0.375
E1 ∧ ¬E2 ∧ E3 ∧ E4	0.9277	0.9277
E1 ∧ ¬E2 ∧ E3 ∧ ¬E4	0.375	0.375
E1 ∧ ¬E2 ∧ ¬E3 ∧ E4	0.8594	0.8594

Figure I.21 Viewing conditional probabilities pane

***Sn* Judgement** May be set in the interval [0,1] and used to calculate the parent Sn value during evidence propagation.

***Sp* Judgement** May be set in the interval [0,1] and used to calculate the parent Sp value during evidence propagation.

This tab displays a fixed length list of all combinations of child evidence, where evidence for is displayed as *E*1, *E*2 *etc.* and evidence against is displayed as ¬*E*1, ¬*E*2 *etc.* There are 2^*N* combinations for *N* children. The conditional probability for the *Sn* and *Sp* cases may be entered separately.

When the tab is enabled the conditional probability values are set to a default using the approximation method which converts necessity and sufficiency to conditional probabilities in the propagation calculations.

The conditional probabilities are lost in the following circumstances:

- If the first tab 'set conditional probability directly' checkbox is unticked
- If the calculation preferences 'enable conditional probabilities' is unticked
- If a link is added or removed from the parent
- If the model is closed

When the tab is enabled the conditional probability values are set to a default using the approximation method which converts necessity and sufficiency to conditional probabilities in the propagation calculations.

I.5.4. Necessity and Sufficiency

Sufficiency is a measure of the amount of influence a given child process has on the performance of its parent. Therefore sufficiency is related to the positive contribution of the child process to the parent.

Necessity is a measure of the extent to which failure of a child process will cause failure of its parent. Therefore necessity is related to failure or poor performance.

These definitions are demonstrated by the following examples (Figure I.22) for the case where a process has a single sub-process which has no uncertainty and defined by the interval [0.5, 0.5].

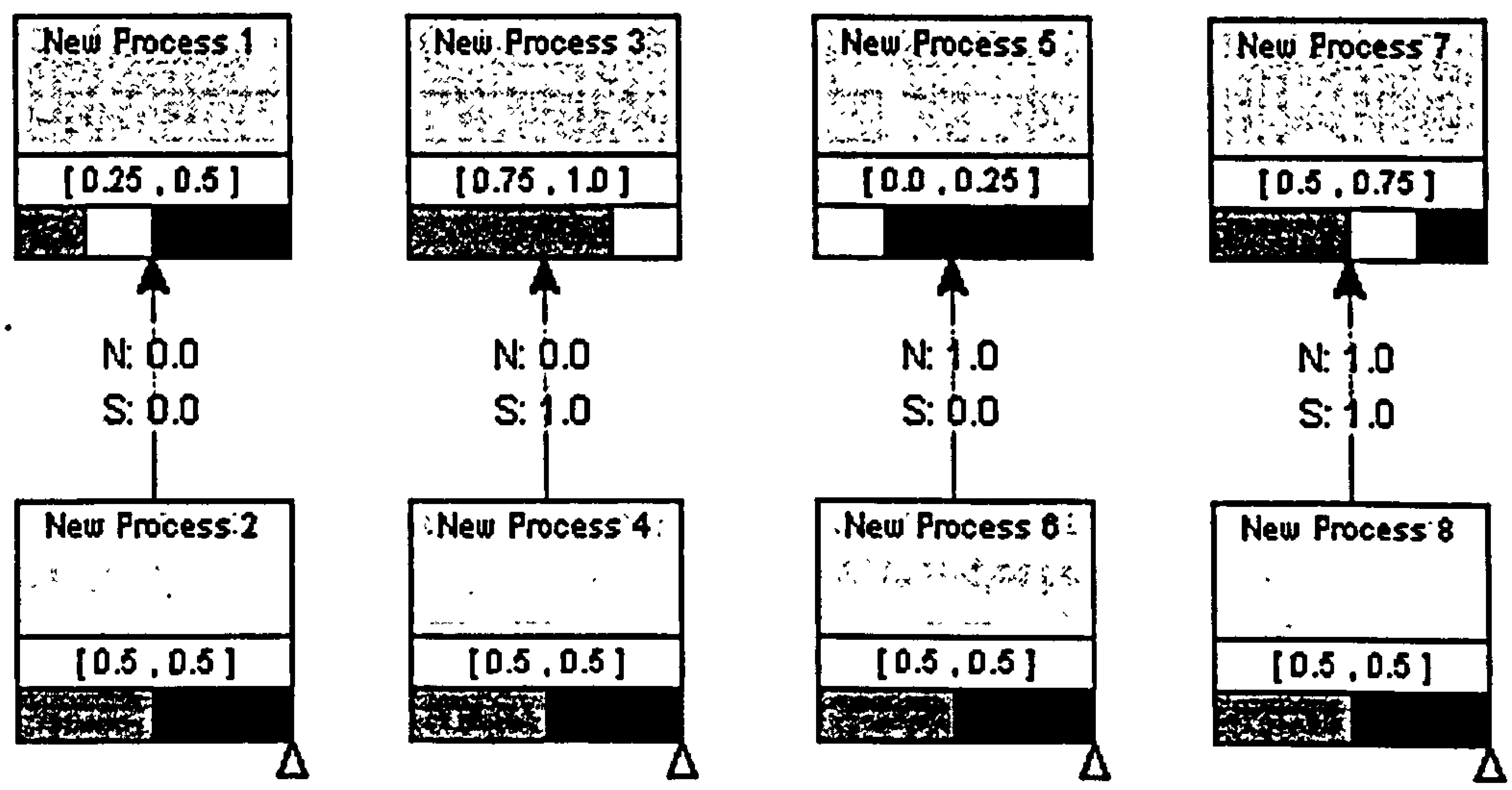

Figure I.22 The influence of 'Necessity' and 'Sufficiency'

(1) In the first case (process 1) necessity and sufficiency are both zero. The interval of the super-system has uncertainty introduced because the relationship between necessity and the interval probability defining the super system is not known precisely (Section H.3).

(2) In the second case (process 3) sufficiency is set to one and the sub-process defines the parent. Evidence for performance is determined by the sub-process and evidence against may vary from 0 to 0.25.

(3) In the third case (process 5) sufficiency is zero and necessity is one. Failure of the parent (evidence against) is determined by the sub-process and evidence in favour of the process may vary from 0 to 0.25.

(4) In the fourth case (process 7) necessity and sufficiency are set to one.

I.5.5. Dependency

Dependency can be interpreted as being due to evidence originating from a common source or being influenced by common processes. The dependency parameter allows uncertain dependency to be added to the PERIMETA model.

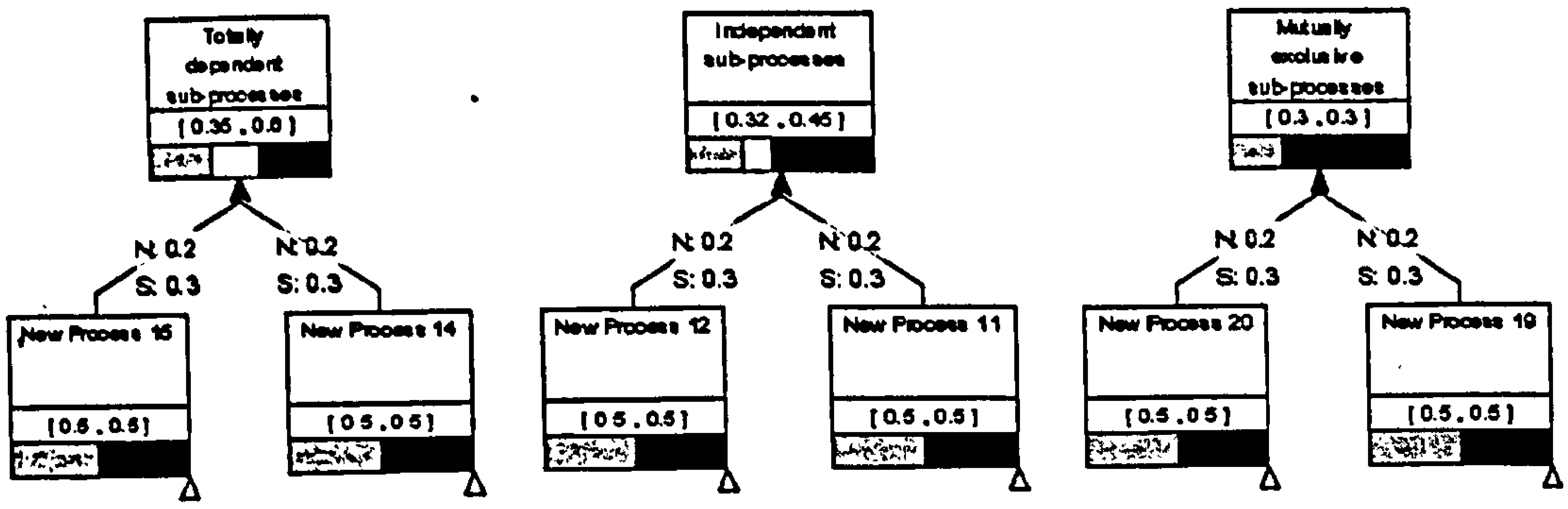

Figure I.23 The effect of changing 'Dependency'

(1) In the first case dependency is set to 1 which represents dependence.

(2) In the second case dependency is set to 0 which represents independence.

(3) In the third case dependency is set to -1 which represents mutual exclusion.

I.6. GLOSSARY OF PERIMETA TERMINOLOGY

Aspect of performance	A view onto system performance which weights specific performance indicators. A default aspect called overview must exist. Further aspects such as safety, cost, operations or environment are optional.
Combined Figure of Merit	A linear weighted sum of local and propagated Figure of Merit for a process.
Conditional probability	User elicited weightings for each combination of child process evidence for and against the proposition. In PERIMETA this is either derived from user entered necessity and sufficiency or may be entered directly for a limited number of child processes.
Dependency	A point value on [-1,1] that defines the relation between 2 sub-processes which share a parent process. -1 represents mutual exclusion, 0 independence, 1 dependence.
Figure of Merit	An interval on [0,1] to indicate evidence for the process proposition (green area), evidence against the proposition (red area) and uncertainty (white area).
Locally measured Figure of Merit	The combined evidence gathered for a specified process, either entered directly or a linear weighted sum of performance indicator evidence.
Necessity	A point value on [0,1] that indicates the extent to which failure of the sub-process will cause failure of the parent system. '1' means that the parent will certainly fail if this sub-process fails, '0' means that it doesn't matter, something else will take its place.
Node	A node in the PERIMETA graph represents a process or sub-process in the engineering system.
Non-dimensional Performance Value	An interval on [0,1] indicating evidence for and against performance and optional uncertainty. Either input directly or derived from a performance indicator.
Performance Indicator	Externally supplied measure of performance, numerical or linguistic with optional uncertainty.
Propagated Figure of Merit	The evidence provided by sub-processes for a parent process. Evidence is propagated using the Interval Probability Theorem.
Sufficiency	A point value on [0,1] that indicates the influence a sub-process has on the performance of the parent process. '1' means no other process is needed, the sub-process is sufficient to define the parent, '0' means the sub-process is irrelevant.
Value Function	Represents organisational objectives and regulatory standards. Used to project a dimensional performance indicator to a non-dimensional value.